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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

MEETING OF WORKING COMMISSION W18 TIMBER STRUCTURES

Technical University
and
Building Research Institute
Copenhagen, Denmark
25-26 October, 1973

2. Sitzung

CONTENTS

1 LIST OF DELEGATES

CANADA	
C K A Stieda	Western Forest Products Laboratory, Vancouver
DENMARK	
T Feldborg	Building Research Institute, Copenhagen
M Johansen	Building Research Institute, Copenhagen
H J Larsen	Technical University, Copenhagen
A R Egerup	Technical University, Copenhagen
P Hoffmeyer	Technical University, Copenhagen
H Riberholt	Technical University, Copenhagen
H P Steenfos	Danish Engineering Institution, Copenhagen
B Lav	Danish Engineering Institution, Copenhagen
ENGLAND	
L G Booth	Imperial College, London
H J Burgess	TRADA, High Wycombe
E Levin	TRADA, High Wycombe
W T Curry	BRE, Princes Risborough
1) A P Mayo	BRE, Princes Risborough
2) J G Sunley	BRE, Princes Risborough
R Marsh	Arap Associates, London
P Steer	Vic Hallam Ltd, Nottingham
FRANCE	
P Crubilé	Centre Technique du Bois, Paris
GERMANY	
H Kolb	Otto-Graf Institut, Stuttgart
HOLLAND	
J Kuipers	University of Technology, Delft
NORWAY	
O Brynildsen	Norsk Treteknisk Institute, Oslo
SWEDEN	
B Norén	Svenska Träforskningsunstitutet, Stockholm

1) - Secretary W18
2) - Co-ordinator W18 and Chairman for meeting

2 CHAIRMAN'S ADDRESS

MR SUNLEY, as Co-ordinator of W18 and Chairman of the meeting, welcomed the delegates to this, the second meeting of the revitalised W18 Group. He said that since the first meeting in March 1973 a considerable amount of interest had been shown in the programme of work which the group proposed to undertake and this was reflected in the membership of the group which now stood at thirty. He went on to say that following the first meeting in March the terms of reference for W18, which had been proposed by the delegates, had been officially accepted by the CIB Programme Committee and these were as follows:

"To study and highlight the major differences between the relevant national design codes and standards and suggest ways in which the future development of these codes and standards might take place in order to minimise or eliminate these differences".

Finally MR SUNLEY outlined the agenda and programme for the two days and this was approved by the delegates.

3 CORRESPONDENCE

MR MAYO, Secretary W18, reported that at the CIB Programme Committee meeting in April 1973, it was generally agreed that W18 and W23 "Basic Structural Engineering Requirements" should form a close liaison and if possible send representatives to each others meetings. Following this Professor A Smirnov, Co-ordinator of W23, wrote to W18 with details of the current work of W23 and a request for similar details of W18. MR MAYO said that in reply he sent a short history of W18 together with an outline of the group's programme of work and an invitation to send a representative to the meeting in Copenhagen. However, no representative of W23 was present in Copenhagen and nothing further had been heard from Professor Smirnov.

MR MAYO continued with a further point from the Programme Committee meeting in April, in which Mr Mathur of the National Buildings' Organisation, New Delhi, suggested that W18 should develop methods of design suitable for the lesser known secondary species of timber and in this context he would provide information on timber resources in hot arrid climates. This information had now been received and the Secretary said that when anyone wished to see it he would provide copies of the papers which were as follows:

a) Secondary Timbers

b) Nailed Joints in Timber Structures Part III

c) Typical Designs of Small Span Timber Trusses Through Modern Timber Engineering Technique

d) Typical Designs of Medium Span Timber Trusses Through Modern Timber Engineering Technique

MR MAYO said that requests had been received for information on specific subjects and on the work of W18 in general, from L'Universite D'Abidjan in West Africa, The Swedish Council for Building Research and the University of Nairobi and copies of the Proceedings of the first meeting of W18 together with an outline of the programme of work had been sent to these organisations.

Finally the Secretary reported that a letter had been received from Mr E Kalkkinen, Director of ECE/FAO Timber Division in which he invited W18 to send a representative to an ad hoc meeting of experts in Geneva on 11-12 October 1973 to discuss the unification of stress grading rules for sawn softwood. Members of W18 had been circulated and finally M P Crubilé of Centré Technique du Bois attended the meeting as the W18 representative.

4 PUBLICATION OF PAPERS

MR SUNLEY raised the question of whether or not technical papers written by members of W18 at the request of the group for presentation at the group's meeting should also be published in the technical journals at the discretion of the author. He said that one advantage of publication could be a greater interest in the work of W18 which in turn would increase the influence which the group was able to exert when putting forward recommendations to both the European and International Standards Organisations. He added that he thought it might also lead to a quicker adoption, by professional engineers, of any recommendations which W18 may make.

MR CURRY replied that he was not in favour of publishing in technical journals those papers which were presented for discussion at the group's meetings. He pointed out that the way in which W18 had chosen to work was by members writing papers on selected subjects for discussion at the meetings. Following discussion it would then be necessary to amend the particular paper in question to take account of the points raised so that the amended paper represented the concensus of opinion of the group. This final paper could then be presented as firm recommendations from W18 and as such, inclusion in the technical journals would, in most cases, be highly desirable. However, MR CURRY thought that if the intermediate papers leading to the final recommendations were also published it would only lead to confusion.

MR LEVIN said he thought that the discussions should be made public in the technical press and that W18 should seek publication of the programme of work in detail in the hope that journals would then request articles on particular topics which W18 should endeavour to provide. This suggestion was not generally accepted by the delegates.

In conclusion MR SUNLEY said that perhaps the most suitable compromise would be for all delegates to receive a summary of each meeting which, at their descretion, they could offer to their own national journals for publication together with their address to which people could write for further information. This suggestion was accepted and agreed by the delegates with the proviso that while stocks last from a limited printing, copies of the Proceedings of each meeting could be available to non-members of W18 with a specific interest in the work of the group.

5 STRESS GRADING OF TIMBER - REPORT ON THE ECE MEETING IN GENEVA, OCTOBER 1973

MR SUNLEY said that the meeting was organised by the ECE/FAO joint committee on timber which invited most of the European experts on stress grading together with representatives from various European and international organisations of which CIB-W18 was one. The objects of the meeting were, he said, to establish which system of grading, if any, each European country used at present and to identify the problems associated with rationalising these systems to produce a unified set of European stress grading rules. MR SUNLEY went on to say that each delegate was invited to submit a statement to the meeting on stress grading in their own country and in addition papers were submitted by:

a) Mr Levin - Timber Research and Development Association, England - Paper on loadings and stresses in use in each country showing the wide variation which exists at present.

b) Professor Thunnell - Swedish Forest Products Research Laboratory - Paper on grading in Sweden.

c) Mr Sunley, Building Research Establishment, England - Paper on the new British Standard BS 4978 'Specification for Timber Grades for Structural Use'.

The meeting went on to consider the obstacles to harmonisation which MR SUNLEY said could really be summed up as a general reluctance to change. Finally a statement was issued which called for a working group on harmonisation to be set up. In general delegates were against this working group being set up within ISO although the Russian delegates thought that ISO should do the work. Finally it was agreed to get the working group to draft a set of "Grading Rules for Europe" which after approval and publication by ECE would be submitted to ISO.

MR LEVIN said that the delegates in Geneva were asked to submit suggestions for the composition of the drafting committee by the 30 November 1973 when it would be formed. It was hoped that the first meeting of this sub-committee could be arranged before the end of the year. MR LEVIN also said that at a meeting of the European Softwood Importers and Exporters on 24 October it was agreed that BS 4978, with some slight modifications, should be recommended to the drafting committee as a basis for the European rules.

MR SUNLEY asked the delegates to the CIB-W18 meeting for comments on how CIB-W18 should be involved with the drafting of the European stress grading rules and DR KUIPERS proposed that CIB-W18 should be represented on the drafting committee. This was agreed and later in the meeting M P Crubilé of Centré Technique du Bois, Paris, was elected as the representative of CIB-W18.

6 REPORT ON MEETING OF IUFRO WORKING GROUP ON STRUCTURAL UTILIZATION IN SOUTH AFRICA, SEPTEMBER 1973

MR SUNLEY reported on the recent meeting of the International Union of Forestry Research Organisations held in South Africa during September. He said that this meeting acted as a general forum on timber engineering in which the research work being undertaken at the present time in each country was discussed. The subjects which were discussed were as follows:

a) Long term testing - This showed that the present factors being used in design were satisfactory for high grade timber but not for the lower grades.

b) Prototype Testing - There appeared to be a move towards more theoretical methods to justify timber designs.

c) Non-Destructive Testing - A number of papers were submitted with the majority dealing with stress grading. A new South African stress grading machine was described and was considered by the delegates to be a useful aid to visual grading. The South African delegates did not agree with this and considered it to be an alternative to visual grading.

d) Methods of assigning stresses to stress grades.

e) Methods of machine stress grading with stress wave techniques.

f) Proof testing.

g) Mechanical properties of timber and the use of statistical methods in the derivation of these properties.

h) Limit state design.

i) Structural safety - Discussion on the different factors of safety used in each country and some recent failures.

j) Design of trussed rafters - Comparisons between theoretical and prototype testing methods of design together with discussions on the design of joints and lateral bracing.

k) Methods of jointing timber - Finger joints, staples and elastomeric adhesives.

MR SUNLEY concluded by saying that he hoped the Proceedings of the meeting would be ready by early 1974.

7 STRUCTURAL DATA

DR BOOTH introduced his paper "The Presentation of Structural Design Data for Plywood" and thanked those members who had provided information to be included. He asked if any members could give information on how the stresses in Table 14 were derived and explained that the stresses issued by the Nordic Building Regulations Committee 1973, Table 15, were derived directly from stresses published in Canada by the Canadian Standards Association and the Council of Forest Industries of British Columbia, see Table 1. The English stresses given in CP 112:L967, see Table 12, were also derived from the same source.

DR KUIPERS in reply to Dr Booth said that the stresses in Table 14 were derived from standard tests conducted in Holland but with the Canadian stresses in mind. He said that the present Dutch code contained methods of testing for plywood and the methods to be used to derive design stresses from the test data obtained.

PROFESSOR LARSEN said that he thought the Dutch testing approach was impracticable due to the large amount of testing required. He also said that the variations with thickness given in Table 16 were unnecessary and DR BOOTH agreed that at the present time this was probably true although he thought that it might not be so in the future when different plywoods were made

There was general agreement that due to the rapid changes taking place at present in the manufacture of plywoods and also the large variability between similar types of plywood (ie Douglas fir plywood), it was not satisfactory to publish design stresses for plywood in design codes as they would quickly go out of date.

MR STIEDA pointed out that the Canadian stresses for rolling shear were only intended for use where plywood was used next to solid timber. He went on to say that generally the equilibrium moisture content for plywood was about 15% whereas it was about 18% for solid timber and he was interested to know how the Canadian stresses, which were based on a moisture content of 15%, were modified to get the English stresses which were based on a moisture content of 18%

MR CURRY in reply said that the method used to modify the Canadian stresses to take account of the change in moisture content was fully described in "A Commentary on the British Standard Code of Practice CP 112:1967 - The Structural Use of Timber" by Booth and Reece.

PROFESSOR LARSEN said he thought stresses should be related to climatic conditions and groupings used for various types of buildings in different climatic conditions instead of stresses being related just to moisture content. He also asked for an explanation of the use of different areas for plywoods of the same thickness in Table 13.

DR BOOTH said this resulted from plywoods having different lay-ups and different nominal thicknesses.

Following the discussion on Dr Booth's paper MR SUNLEY asked each delegate to describe the present position in their own country regarding the specification of stresses for plywood and the way they would like to see it develop.

DR NOREN said that the Scandinavian countries would be working with characteristic stress values in the near future and there were two methods of giving stresses for plywoods.

a) Plywood could be manufactured to a known specification which would be tested and stresses published for plywood made only to that specification.

b) Have a range of stresses related to some emperical methods of testing.

He said that if there were only a few types of plywood, method (a) was preferable, but if there were many types he thought it better to specify stresses for three grades, P_{20} P_{30} and P_{40}, and test each type of plywood and assign it to a particular grade. The stresses associated with these grades will be based on the parallel plies approach and related to the ultimate strength in bending. The average stresses for each grade would be given in order to accommodate different thicknesses of plywood and it would be necessary to know the lay-up of the plies.

PROFESSOR LARSEN said he did not think plywood stresses should be included in a structural code but that characteristic stresses related to the full cross-section should be published separately. He said he would also like to see design data presented in the form of products of stresses and the section properties, ie Tables of FA, EA, FZ, and EI.

MR EGERUP said that while he agreed with Professor Larsen he did think that values of E and I should be published as well as the products.

MR BRYNILDSEN said that the Norwegian code did not contain plywood details and it was left up to the manufacturers to produce and publish design data. He went on to say that he would prefer to adopt the NKB system which basically was what Professor Larsen had described earlier.

DR KUIPERS said that the Dutch timber code did not contain plywood details but that there was a separate code on methods of test for plywood in which design data was presented in the form of tables and graphs.

Dipl Ing KOLB said that the structural use of plywood in West Germany was not large and the present standard, which gave design stresses related to the full cross-section area, was out of date. At present only one type of plywood is allowed for structural use and this is manufactured to a strict specification and identified with a special marking.

M CRUBILÉ said that in France the principle authorities tend to deal with plywood on a parallel plies approach but engineers in industry preferred to work with full cross-section values.

MR STIEDA said he could not foresee any significant changes in the methods used to specify plywood and stresses in Canada at present and these were accuately described in Dr Booth's paper.

DR BOOTH said that the new British Code of Practice which is at present under revision will specify stresses for Douglas fir and Finnish birch plywoods in tabular form and that other plywoods would be assessed or the properties of the individual plies with some method, yet to be devised, of combining the properties for each ply.

MR LEVIN said he thought timber codes should give methods for assessing stresses for different plywoods but should not specify the actual stresses. These should be published by the manufacturers.

MR STEER said that in Britain at present there was a shortage of plywood and new types, including hardwoods, were being introduced. He said that difficulties were being experienced with testing to derive design stresses for these new plywoods and to ensure adequate quality control. On the grading of plywoods, as described by Dr Noren, he thought that this would be wasteful as in general the grade to which a particular plywood was assigned would be determined by the weakest property when for some uses a different property, appropriate to a higher grade, may be more important.

MR STIEDA asked what the views of the delegates were on the different methods of testing.

DR BOOTH said that the current British Standard on testing of plywood dealt with small sized samples but he thought these were only satisfactory for comparison purposes and the tests at present in use for large sized samples or full boards were being assessed.

DR KUIPERS said that in Holland some larger size specimens were tested, ie 6 inch wide samples for tension test, but in the main small sized samples were used for testing.

DR NOREN said that in Sweden for quality assessment purposes large sized samples were used as well as small ones, but tests were also carried out on a performance standard basis.

In conclusion MR SUNLEY said that there appeared to be a number of ways of presenting design data for plywood and no country had any definite proposals to change the particular method in use there at present. There was an MKB system in Scandinavia based on the strength and stiffness method of presentation, but Sweden was not entirely in agreement with this system although Norway would adopt it if the use of structural plywood became sufficiently large.

The current revision of the British code will probably not use the strength and stiffness method but will be based on a modified parallel plies approach based on stress. In addition the same information would be presented elsewhere at a lower level in the form of span tables and charts.

Finally it was agreed that Mr Curry should draft a statement to sum up the present position. This statement was subsequently agreed by the delegates and is as follows.

> "Where there exists an adequate specification and quality control procedure for a particular commercial plywood the strength properties should be determined from a programme of laboratory tests on samples covering the range of constructions produced from which characteristic strength values should be derived and presented in terms of the strength and stiffness of a standard width of section.
>
> It is however recognised that because of the fluid position regarding the manufacture of plywood and the wide range of species and constructions which could become available, a code should also make provision for the use of predictive methods based on limited laboratory tests or alternatively on a knowledge of the strength characteristics of the species used".

8 INTERNATIONAL CODE OF PRACTICE FOR STRUCTURAL TIMBER

MR CURRY introduced his paper "A Framework for the Production of an International Code of Practice for the Structural Use of Timber" as the basis for a programme of work for CIB-W18. He proposed that members should write detailed papers on items selected from the framework and he mentioned the previous paper on plywood by

Dr Booth as a good example. Following the discussion of these individual papers by CIB-W18 a recommendation should be drafted for each heading in the proposed framework and it would then be left to individual countries to draft the actual design clauses.

Commenting on the proposed framework, MR EGERUP said that he thought panel board and fibre board should be included and MR LEVIN agreed together with Professor LARSEN.

MR STIEDA asked what information a code based on the framework would include on material requirements and also whether it would contain loading requirements specifically for timber, or would the loadings specified in other codes, eg those published by ISO, be adopted.

MR CURRY in reply said that such a code would refer to material specifications such as the British Standard for stress graded timber, BS 4978 and it would not include loading details.

MR LAV said that when Denmark changed to the limit state method of design, difficulties arose because some of the design loads included safety factors as did some of the design stresses and therefore it was very difficult to assess the true safety factor on any particular design.

MR STIEDA said that load factors for limit state design had already been agreed on an international basis for concrete and in Scandinavia the NKB had recommended that these load factors should be adopted for all materials including timber. In Sweden, however, there was a possibility that these load factors may be modified slightly.

DR BOOTH said he did not see why the load factors needed to be the same for all materials as it was quite possible that in the light of experience the factors for concrete would be changed and hence the factors for timber would also have to change.

MR SUNLEY agreed with Dr Booth, but he did not think that timber would be allowed to have special factors different from other materials and in which case efforts would be made to make adjustments elsewhere in the design to allow the concrete factors to be used.

Commenting on Mr Curry's paper, Professor LARSEN said that he did not agree that changes in design procedure should not be allowed to incur a penalty on section sizes (Page 1, para 3). He thought that the lack of timber failures due to design inadequacies may be due to hidden load factors inherent in the present methods of design which may be excluded with a different method of design. However, Professor Larsen's view was not supported by the other delegates.

Professor LARSEN continued by saying that he considered the modification factors for load sharing on page 4 to be design factors and not material factors and therefore they should be included elsewhere. Also on page 9 - section b) Quality Control should only refer to other publications and specifications and should not itself contain details. Finally, on page 5, third paragraph, Professor LARSEN did not think it was necessary to refer specifically to Finland and British Columbia as being the major sources of plywood.

DR KUIPERS said that in respect of laboratory tests on plywood and joints the code should only contain references to other publications where these tests would be described in detail. This was agreed provided the methods of test and the methods of analysing the test data were internationally agreed. Further it was not thought that CIB-W18 should be responsible for developing these methods and that RILEM would be better suited to do this work. However, it was suggested that both Finland and Canada were developing test methods for plywood and if possible RILEM should accept the methods proposed by these countries.

DR KUIPERS questioned the exclusion from the code of all timber connectors other than nails, screws and bolts - see page 6.

In reply DR BOOTH said that such an exclusion was necessary because of the large variety of other connectors available, but these connectors would be catered for by including in the code special test methods which anyone including the manufacturers could apply to derive the necessary design data.

DR KUIPERS said that this would only be satisfactory if the tests conducted by the manufacturers were carried out correctly and accepted by designers and specifiers.

Finally Professor LARSEN said that along with this code framework, it would also be desirable to rationalise the notation used in the work of CIB-W18. This was agreed and DR KUIPERS and DR NOREN agreed to write a paper on notation for the next meeting of CIB-W18.

9 SOLID TIMBER COLUMNS

Professor LARSEN introduced his paper "The Design of Solid Timber Columns" which included a survey of the methods used in different countries to design timber columns. He concluded his introduction by saying that he thought the Dutch approach had a lot to commend it over other methods, although he thought the eccentricities which were used were too high and there should be no deflection limitations at the ultimate stage.

DR NOREN questioned whether formula (12) could be used for long term loading. Professor LARSEN replied that the current theories dealing with long term loading included a number of parameters which at present were unknown and therefore formula (12) was the best available to the practising engineer.

MR SUNLEY said he thought the United Kingdom could adopt the Dutch method without too much difficulty, although the timber species would have to be grouped into possibly six groups with average stresses assigned to each group.

MR STEER agreed with this view although he pointed out that in Britain the sum of the combined stress ratios is limited to 0.9 instead of 1.0 as in Holland and other countries.

MR SUNLEY proposed that the Dutch method with some modifications should be adopted by the group as the basis of a method to be included in a European code and he requested Professor Larsen to write a further paper for the next meeting, extending the method to spaced columns. This was generally agreed by the delegates and Professor LARSEN agreed to write a further paper.

10 FUTURE PROGRAMME OF WORK

In opening the discussion on the future programme of work of W18 MR SUNLEY read out the terms of reference of the group which are as follows:

"To study and highlight the major differences between the relevant national design codes and standards and suggest ways in which the future development of these codes and standards might take place in order to minimise or eliminate these differences".

He then asked for comments on the terms of reference and also what delegates thought ought to be the relationships between W18 and other organisations such as ISO.

MR STEENFOS said he thought W18 should develop a good liaison with the relevant technical committees in ISO and should look at the papers and articles already with these ISO committees and try and form the proposed new European code of practice from them.

It was generally agreed that a good relationship should be established particularly with the ISO committee TC 98.

MR LEVIN asked if it was known whether or not TC 98 proposed to issue codes of practice. In reply MR STEENFOS said that this would not be the case and that TC 98 would only deal with the basic principles.

MR LAV said he was concerned that W18 might be duplicating work already being carried out by other organisations such as CEN which as he understood proposed to take the documents issued by ISO and modify them to suit Europe.

Following on from this discussion MR SUNLEY asked for subjects for the next meeting and it was agreed that the following articles and papers would be submitted.

a) Predictive methods for the derivation of stresses for plywood by C K A Stieda

b) A comparison of present methods of test for plywood by J Kuipers

c) Spaced columns by H J Larsen

d) Symbols and notation by J Kuipers and B Norén

e) Load sharing factors by E Levin

f) A report on the progress of the ECE stress grading draft by P Crubilé

g) Long term loading by B Norén

In conclusion MR SUNLEY informed the meeting that DR KUIPERS had kindly offered to act as host for the next meeting which would be held in Delft in June 1974. Finally, on behalf of all the delegates he thanked Professor Larsen for organising the present meeting and for the hospitality offered to delegates by Professor Larsen and his colleagues at the Technical University and by Mr Hohansen and his colleagues at the Building Research Institute

11 PAPERS PRESENTED TO THE MEETING

PAPER 1

The Presentation of Structural Design Data for Plywood. By L G Booth

PAPER 2

A Framework for the Production of an International Code of Practice for the Structural Use of Timber. By W T Curry

PAPER 3

The Design of Solid Timber Columns. By H J Larsen

PAPER 1

CIB - WORKING COMMISSION W18

THE PRESENTATION OF STRUCTURAL DESIGN DATA FOR PLYWOOD

by

L G BOOTH

DEPARTMENT OF CIVIL ENGINEERING
IMPERIAL COLLEGE - LONDON

COPENHAGEN - OCTOBER 1973

Contents

1 Introduction

The purpose of this paper is to review the various methods of presentation of structural design data for plywood that have been used in North America and Europe.

In North America it has been the tradition to present design stresses for plywood on the assumption that those plies that are perpendicular to the direction of the stress contribute little to the strength of the panel and that a good approximation can be achieved by considering "parallel plies only". In contrast, the majority of European countries have ignored the layered construction of the material and have derived design stresses that are applicable to the "full cross-sectional area". The United Kingdom, with its early dependence on North American supplies of structural plywood, initially adopted the "parallel plies only" approach, but with an increasing supply of Finnish birch plywood, which was specified on the full cross-section, later changed to the full cross-sectional approach and consequently needed to transform parallel plies only Canadian Douglas fir stresses to equivalent full cross-sectional area stresses: however, the United Kingdom may now revert to some form of presentation based on the properties of the individual veneers.

During discussions both in Europe and in North America it has been apparent to the author that this particular topic invokes feelings of almost religious intensity. Much of the discussion is prejudiced and is often based on no knowledge of the "opposition's" point of view. Indeed, it often appears that timber engineers are placed at birth in plywood cradles which rock them to an immediate certainty that "parallel plies only" or "full cross-section" is to be their life's belief.

It is not intended that this report should reach any firm conclusions. It is however hoped that it will enable timber engineers to see some other points of view. The theoretical backgrounds of the methods are briefly discussed and their relative merits compared. The application of the design data is illustrated in an Appendix to this report which gives outline designs of some typical structural components.

2 Units and nomenclature

The emphasis of this paper is on the method of presentation of the design data and consequently the numerical values given in the various tables are not important. No attempt has therefore been made to convert the figures to the same units: it will be found that Imperial and metric units are used.

The tables of design stresses are not immediately comparable, even if translated to the same units. The duration of load and moisture content base for each table is usually different: some stresses are "working", some are "characteristic".

The nomenclature for the stresses varies and has not been altered for the tables. In the text of the report the term "design" has been used to cover all stresses used in calculations, whether they are at a working or characteristic level. Similarly, "design strength" embraces the resistance to moments and forces, and stiffnesses.

The term "geometrical properties" has been used to cover the veneer thicknesses, first and second moments of area of unit widths of panels, etc.

3 Parallel plies only approach

The "parallel plies only" design procedure was first suggested by the United States Forest Products Laboratory at Madison in reports by Freas (1942) and Liska (1942). It was argued that since plywood consists of three or more veneers glued together with the grain of alternate veneers at right angles to each other, it should be analysed as a layered material. In such a compound member subjected to stress, the proportion of the total load carried by the individual layers depends on the moduli of elasticity and the thicknesses of the layers. Although veneers of the same species will be of equal strength (subject of course to variability), the strength of a layer will depend on the angle of the load to the grain of the veneer. By assuming that the material is linearly elastic to failure it is therefore possible to predict the behaviour of plywood under stress, provided the properties of the individual veneers are known.

In the case of tensile forces, since the tensile strength of timber parallel to the grain may be as much as 40 times the strength perpendicular to the grain, the tensile load on a piece of plywood will be carried predominantly by the veneers parallel to the load. An approximate estimate of behaviour could be obtained by assuming the section consists of only the veneers parallel to the stress. If this approach is adopted it may be argued that plywood is not a compound material, but consists of pieces of solid timber stressed parallel to the grain, in which case it is appropriate to use the same basic stresses for plywood as are used for solid timber. This is the parallel plies only approach that was suggested by the United States Forest Products Laboratory.

Tests by Freas (1942 and 1956) and Liska (1942 and 1955) showed that, under certain circumstances, the approximation is very good, with the behaviour in tension and compression being predicted better than that in bending. The approximation is better with thicker than thinner plywood, and it is also better in bending when the outer plies are parallel to the span, rather than at right angles to the span. The approximation is consequently at its worst when 3 ply is used in bending with the outer ply perpendicular to the span; in this case the inner ply is near to the neutral axis and has little opportunity of contributing to a total strength mainly derived from the face veneers. The actual moment of resistance is about 50 per cent higher than that predicted by the approximate theory. Consequently the theory has to be modified by factors which depend on the direction of the face ply with respect to the span and the number of plies.

In the case of bending the procedure initially adopted was to give only the basic stress in bending and to give in addition a list of modification factors for different grades, numbers of veneers and direction of face grain with respect to the span. This procedure has now been abandoned and the modification factors have been incorporated in the stresses, with the result that the design stresses now depend on the number of veneers. These stresses are however still applicable to the parallel plies only approach.

The idea of calculating the stress based on parallel plies only fails when the stress is at 45° to the face ply, since in a tension or compression member all the plies have the same stress and the calculation of the stress must be based on the full cross-section. Similarly for panel shear the stress must be calculated on the full cross-section.

Although the original idea of having the same basic stresses for solid timber and plywood is attractive, it in fact results in a design procedure which uses both the parallel plies only and the full cross-sectional approaches, together with a number of hidden modification factors to bring the theory in to agreement with experimental evidence.

A knowledge of the lay-up of a plywood is necessary for all stress calculations, and the task of the designer is made easier if the geometrical properties (A, I, etc) are tabulated for the thicknesses and lay-ups that are currently available.

The Canadian Code CSA 086 (Canadian Standards Association, 1970) is based on the parallel plies only approach and gives stresses for a number of grades, lay-ups and geometrical properties. Similar information is published by the Council of Forest Industries of British Columbia (COFI, 1972) and this data is given as Tables 1 and 2 of this report.

The parallel plies only approach is adopted throughout the United States of America and the tables of design stresses are similar to those published in Canada, but for more combinations of species and grades (American Plywood Association, 1966).

4 Full cross-sectional area approach

4.1 Design stresses

In Scandinavia design stresses for plywood have traditionally been presented for use with sectional properties based on the full cross-section. Early examples of this approach are by Niskanen (1963) and Noren (1963 and 1964).

Noren (1964) presented a theory for the ultimate strength of plywood in which the deviation from Hooke's law due to flow in veneers subjected to high compression stresses was incorporated. He showed that in certain cases the veneers which have their grain direction perpendicular to the direction of the principal stresses may be neglected without much loss in accuracy. Although this was in effect an advanced form of the parallel plies only approach the design stresses were based on the full cross-section.

Niskanen (1963) ignored the fact that plywood is a layered material made up from veneers with known properties. He assumed that it was a new material and investigated its strength properties by carrying out tests on small specimens and built-up components. He found that its strength depended on the number of plies and on the direction of the stress with respect to the face grain, and from the test results the ultimate stresses were based on the full cross-section.

Although the stresses are applicable to the full cross-section, a knowledge of the lay-up of the plywood is still required for some stress calculations. Again the task of the designer is made easier if geometrical properties are tabulated for currently available plywoods.

In the United Kingdom the full cross-sectional area approach was adopted for the 1967 edition of the Code of

Practice CP 112 (British Standards Institution, 1967) and the design stresses and geometrical properties are given in Tables 3 and 4 of this report.

In the Netherlands the full cross-section has also been adopted and Tables 5 and 6 show a slightly different method of presentation of the design data (Vermeyden, 1967).

In Germany DIN 1052 (Deutsche Normen, 1969) presents stresses for the full cross-sectional area for plywoods manufactured to DIN 68705, part 3: geometrical properties are not given. An extract from DIN 1052 is given in Table 7.

4.2 Design strength

It was noted above that Noren (1964) derived design stresses for Swedish pine plywood. These stresses were presented in a table which also gave the permissible bending moment, tension and compression forces per unit width of panel. The design stresses were applicable to the full cross-section and the design moments and forces were derived for the full cross-section (see Table 8).

Noren's method of presentation has been extended by Larsen (1971) to include the bending and direct force stiffnesses per unit width of panel (see Table 9).

5 Transformation of design stresses and moduli of elasticity from parallel plies only to full cross-sectional area

5.1 Introduction

Much of the test data on Douglas fir plywood comes from North America and the derived stresses are applicable to the parallel plies only. If this data is to be used with the full cross-section, equivalent stresses must be calculated. This transformation will be discussed in this report in relation to the approach adopted in the United Kingdom: it would appear that the same approach has been used elsewhere in Europe.

5.2 Transformation used in the UK

In the United Kingdom the first edition of CP 112 was published in 1952 and it contained design data for only Douglas fir plywood manufactured in the United States. No test data was available in the UK and the Code adopted American stresses and their parallel plies only method of presentation.

The revision of CP 112 (1952) began in 1956 and after an elephantine period of gestation was finally published in 1967. During this period American Douglas fir became uneconomical and structural engineers began to use either Canadian Douglas fir or Finnish birch. The policy of the Code committee was to accept the test data from Canada and Finland, and to seek the recommendations of Forest Products Research Laboratory, Princes Risborough on design stresses. The data from Canada was based on parallel plies only, whilst that from Finland was Niskanen's work, which used the full cross-section. The Committee decided that to tabulate the stresses for Douglas fir on parallel plies only, and for Finnish birch on the full cross-section, would cause confusion and that one method should be adopted. The relative advantages and disadvantages of the two approaches will be discussed later in this paper and it is sufficient to say here that the full cross-sectional area approach was adopted for both plywoods in CP 112 (1967).

The data for Finnish birch plywood was already based on the full cross-section, but the parallel plies only stresses for Douglas fir needed to be transformed into equivalent stresses for the full cross-section.

The design stresses used in Canada are those given in Table 1, and they are applicable to normal loading and a moisture content of 15 per cent. An initial transformation was made to a base of long-term (permanent) loading and a moisture content of 18 per cent, and the starting point for the transformation from parallel plies only to the full cross-section was therefore the stresses given in Table 10. The method of transformation was as follows (Hearmon, 1965) and later in this report some inconsistencies in this method will be discussed.

Bending stress

For laterally loaded plywood the transformation of bending stresses was made by equating the moments of resistance M. For the parallel plies only

$$M = \frac{2\sigma_{//}I_1}{t} \quad \text{for face grain parallel to the span}$$

and $$M = \frac{2\sigma_{//}I_2}{t - 2a_f} \quad \text{for face grain perpendicular to the span}$$

where I_1 = second moment of area of plies parallel to the span

I_2 = second moment of area of plies perpendicular to the span

t = total thickness of section

a_f = thickness of face ply

$\sigma_{//}$ = grade stress given in Table 10

On a full cross-section basis

$$M = \frac{2\bar{\sigma}I}{t}$$

where I = second moment of area of total section = $I_1 + I_2$

$\bar{\sigma}$ = grade stress on "full cross-section" basis

Hence $\bar{\sigma} = \dfrac{\sigma_{//}I_1}{I}$ for face grain parallel to span

and $\bar{\sigma} = \dfrac{\sigma_{//}I_2 t}{I(t-2a_f)}$ for face grain perpendicular to the span

<u>Direct stress</u>

For plywood subjected to tensile or compressive stresses, the equations of transformation used were

$\bar{\sigma} = \dfrac{\sigma_{//}A_1}{A}$ for stress parallel to face grain

$\bar{\sigma} = \dfrac{\sigma_{//}A_2}{A}$ for stress perpendicular to face grain

where A_1 = area of plies parallel to face grain

A_2 = area of plies perpendicular to face grain

A = total area of plies = $A_1 + A_2$

Bending stiffness

The relationship between the moduli of elasticity in bending was found from the equation

$$\bar{E}\,I = \sum_{i=1}^{i=n} E_i I_i$$

where $\bar{E}$ = modulus of elasticity in bending

E_i = modulus of elasticity of i^{th} ply

I_i = second moment of area of i^{th} ply

It was assumed that the modulus of elasticity perpendicular to the grain (E_2) was 1/20 x modulus of elasticity parallel to the grain (E_1) then,

$\bar{E}\,I = E_1(I_1 + I_2/20)$ parallel to the face grain

and $\bar{E}\,I = E_1(I_2 + I_1/20)$ perpendicular to the face grain

Direct stiffness

Similarly for plywood in tension or compression

$\bar{E}\,A = E_1(A_1 + A_2/20)$ parallel to the face grain

and $\bar{E}\,A = E_1(A_2 + A_1/20)$ perpendicular to the face grain

where $\bar{E}$ = modulus of elasticity in tension or compression

The remaining types of stress were already computed on the full cross-section and consequently no transformation was required.

By the method given above, the parallel plies only design stresses for Douglas fir given in Table 10, in conjunction with the geometrical properties given in Table 11, were transformed into the full cross-sectional area stresses of Table 44 of CP 112 (1967). Table 44 gave design stresses for a number of grades of plywood but for the purpose of this paper Table 12 has been prepared which gives the design stresses for only unsanded sheathing grade: the corresponding geometrical properties of the full cross-section are given in Table 13.

A somewhat similar table of design stresses for Douglas fir using the full cross-section approach has been published in the Netherlands (Vermeyden, 1969) and is given in Table 14.

5.3 Proposed method of transformation

It was mentioned above that some inconsistencies arise from the method of transformation adopted in the UK. These will now be discussed.

In the United Kingdom the best literature on the design of plywood structural components is published by the Council of Forest Industries of British Columbia, which has issued many Canádian publications using parallel plies only design methods. If we modify the Canadian design stresses (Table 1), which are for normal loading at 15 per cent moisture content, to give long-term loading stresses at 18 per cent moisture content we obtain the corresponding UK design stresses (Table 10): the UK stresses were then used in the transformation described above to give equivalent full cross-section stresses (Table 13). Since Tables 10 and 13 are meant to be equivalent it is desirable that structural components designed by the two approaches should be the same: this is not the case.

In the Canadian design practice the contribution of the perpendicular plies is ignored under direct forces and for the majority of lay-ups when bending deflections are being calculated. If we equate the strength of the plywood and its deformations under load the equations of transformation now become:

Bending stress

Face grain parallel to the span

$$\frac{2\sigma_{//}I_1}{t} = M = \frac{2\bar{\sigma}\ I}{t} \qquad \bar{\sigma} = \frac{\sigma_{//}I_1}{I}$$

Face grain perpendicular to the span

$$\frac{2\sigma_{//}I_2}{t - 2a_f} = M = \frac{2\bar{\sigma}\ I}{t} \qquad \bar{\sigma} = \frac{\sigma_{//}I_2 t}{I(t-2a_f)}$$

Direct stress

Face grain parallel to the span

$$\sigma_{//}A_1 = P = \bar{\sigma}A \qquad \bar{\sigma} = \frac{\sigma_{//}A_1}{A}$$

Face grain perpendicular to the span

$$\sigma_{//}A_2 = P = \bar{\sigma}A \qquad \bar{\sigma} = \frac{\sigma_{//}A_2}{A}$$

It can be seen that for bending and direct force the above equations of transformation are as before. For the remaining properties different equations arise

Bending stiffness

$$\sum_{//} E_i I_i = \bar{E}\ I$$

where the summation is taken over parallel plies only.

For face grain parallel to the span

$$E = \frac{1}{I} \sum_{//} E_i I_i = \frac{E_1 I_1}{I}$$

For face grain perpendicular to the span

$$\bar{E} = \frac{E_1 I_2}{I}$$

Direct stiffness

$$\sum_{//} E_i A_i = \bar{E} A$$

where the summation is taken over parallel plies only.

For face grain parallel to the span

$$\bar{E} = \frac{1}{A} \sum_{//} E_i A_i = \frac{E_1 A_1}{A}$$

For face grain perpendicular to the span

$$\bar{E} = \frac{E_1 A_2}{A}$$

For the stiffnesses, the differences between the transformed values are large for thin plywood and decrease as the thickness of the plywood increases. For a typical structural thickness (say 1/2 in) the differences are less than 4 per cent.

The stresses in compression perpendicular to the grain and panel shear are both computed over the full cross-section in the parallel plies only approach and no transformation is required. Similarly the rolling shear stresses are based on the full cross-section, but in this case a transformation is required if these stresses are caused by bending and this may be found by equating the shear resistance of the section.

Rolling shear

$$\tau_{//} \frac{\sum_{//} I_i}{\sum_{//} Q_i} = V = \frac{\bar{\tau}\, I}{Q}$$

where $\tau_{//}$ = permissible rolling shear stress for parallel plies only

$\sum_{//} Q_i$ = first moment of area above critical shear plane taken over parallel plies only

$\bar{\tau}$ = permissible rolling shear stress for full cross-sectional area

Q = first moment of full area above critical shear plane

For face grain parallel to span

$$\bar{\tau} = \tau_{//} \frac{Q}{Q_1} \frac{I_1}{I} = k_1 \tau_{//}$$

For face grain perpendicular to span

$$\bar{\tau} = \tau_{//} \frac{Q}{Q_2} \frac{I_2}{I} = k_2 \tau_{//}$$

The values of $\bar{\tau}$ for Canadian Douglas fir plywood have been incorporated in Table 16. It can be seen that the values of k_1 vary from 1.05 to 1.25, and of k_2 from 0.65 to 0.97 (ignoring the infinite values). Hence when the face grain is parallel to the span, the use of the same rolling shear stress for all lay-ups will underestimate the shear resistance by up to 25 per cent, but when the face grain is perpendicular to the span the strength will be overestimated by up to 35 per cent.

If these changes are made in the method of transforming the stresses, plywoods will have the same shear strength according to both approaches. When plywood is combined with solid timber (for example, stressed skin panels and box beams) some inconsistencies may still arise: these will be noted in the illustrative examples of the design of several components according to different methods which are given in the Appendix to this report.

6 Transformation of parallel plies only design stresses to design strengths

In section 4.2 of this report a tabular form by Larsen (1971) for the design strength of Finply was mentioned and was illustrated in Table 9. In that case the design strengths were derived from full cross-sectional area stresses, but the same tabular form may be used for plywoods whose design stresses are based on parallel plies only. A design strength table for Douglas fir plywood is given in the Code for Specification of Strength and Stiffness Values for Wood Based Boards which has been drawn up by the Structural Timber Group of the Nordic Building Regulations Committee (see Table 15) and it is assumed that this table is derived from Canadian parallel ply stresses. Table 15 gives a constant value for the rolling shear strength and will produce the same inconsistencies that were mentioned in section 5.2.

This discrepancy can be overcome by extending the table to give the shear strength when governed by rolling shear caused by bending. For rolling shear in gusset plates a constant stress is applicable.

Table 16 illustrates this format for Canadian Douglas fir plywood and the appropriate values for 1/2 in unsanded sheathing grade have been calculated, together with the modified rolling shear stresses for all thicknesses.

7 Discussion

For the purpose of the discussion it is better to change the order of presentation of the previous part of this report, which was determined by historical reasons, and to reverse the order and consider design strengths, and finally design stresses.

7.1 Design strengths

We may divide the structural uses of plywood into two main categories: first, plywood acting alone (for example, as flooring laid over joists) and secondly, plywood acting structurally in conjunction with solid timber (for example, stressed skin panels, box beams).

In the first case the design does not require the services of an engineer and can be achieved by making available load-span tables or graphs in either the Code of Practice or, probably better, in plywood manufacturers' design data (Graph 1). If it is not the policy to include load-span data in the Code, then to complete a design the plywood only requires specification in terms of its design strength (moments of resistance, stiffnesses, etc) in any of the ways shown in Tables 9, 15 and 16.

The specification by design strengths eliminates the need to specify stresses but the lay-up must be specified and the design strength will only be appropriate for that lay-up. Designs may be done quicker than with design stresses, although whatever method is used it is not a long process and the saving in time is small: psychologically it encourages the use of a material if load-span tables are available. A final advantage is that it eliminates the need to make a choice of the parallel plies only or the full cross-section approach.

In the second case (timber plus plywood components), standard designs are inappropriate for a Code and the design engineer requires basic information. The preparation of designs using design strengths only is not always possible (eg the calculation of maximum plywood shear stresses in a stressed skin panel) and it is probably more convenient to work with design stresses.

7.2 Design stresses

If the argument that the design of engineered components is better undertaken with stresses is accepted, then the Code is faced with making a decision on whether to specify stresses on the full cross-section or on parallel plies only (or some alternative based on veneer strengths).

7.2.1 Full cross-section

If a plywood of a particular combination of species and geometry has been adequately tested then the most convenient method of specifying the stresses is on the full cross-section. The actual calculation of either ultimate or design stresses, however, implies the adoption of some theory for behaviour under load such as Curry (1953, 1954, 1957a, 1957b), Noren (1964) and Rautakorpi (1971) have proposed. The design stresses are appropriate for only the lay-ups tested and a change of lay-ups by the manufacturer entails a new programme of testing.

Although the stresses are appropriate for calculations based on the full cross-section, the designer needs to know the lay-up of the material and must be able to calculate shear stresses within a layered material.

7.2.2 Parallel plies only

Adequate testing on the scale implied in Section 7.2.1 is not always available. This may be the case for a number of reasons: a change of veneer thicknesses (recent Canadian experience), a change of species (birch to birch faced plywood in Finland), the development of a new industry (in Sweden, Malaysia, etc). In these cases it is desirable to have within a Code some method by which design stresses can be predicted from a knowledge of the lay-up of the plywood. Such a method is always likely to be less efficient than full-scale testing and must incorporate factors of safety which err on the safe side: under these circumstances it is doubtful, bearing in mind the variability of the material, if a complex theory (eg Rautakorpi, 1971) is worthwhile in comparison with a simple, but not so accurate theory such as parallel plies only.

It is however not the purpose of this report to discuss the various methods proposed for the prediction of strength (Markwardt and Freas, 1946: Curry, 1953, 1954, 1957a, 1957b: Niskanen, 1963: Noren, 1964: Rautakorpi, 1971) but if the argument developed above is accepted, then this decision must at some stage be made. Discussion of the parallel plies only approach is appropriate in this report from the point of view of a method of presentation and use, but not prediction of behaviour.

In the case of plywood acting alone, a design based on parallel plies only stresses is probably too difficult for the non engineer. The need for these design calculations can be eliminated by the preparation of load-span tables that can easily be used by the non specialist (Graph 1).

When plywood acts in conjunction with solid timber (stressed skin panels, box beams, etc) the design calculations should be undertaken by an engineer and there will be no great difficulty, after an initial explanation, for him to understand the behaviour of a layered material with stresses specified on the parallel plies only. It was pointed out above that with full cross-section stresses a knowledge of layered material behaviour is required to calculate shear stresses in a plywood skin. In the case of parallel plies only this approach is extended to all parts of the component. It could be argued that an advantage of using parallel plies only is that the engineer is forced to think in terms of a layered material throughout the calculation, and he automatically calculates the critical shear plane in a stressed skin panel, whereas the use of the full cross-section approach hides the nature of the material and can lead to the newcomer's failure to calculate the critical stresses.

The comments in the last paragraph apply to the <u>use</u> of the stresses. From the point of view of <u>presentation</u>, <u>pure</u> parallel plies only stresses would be superior to full cross-section stresses because they would not depend on the number of veneers, but it can be seen (Table 1) that the parallel plies only stresses do depend to a small extent on the number of veneers. A major advantage put forward by the proponents of parallel plies only is that the design stresses are independent of the lay-up and that, for example, the lay-ups in Table 2 may be changed by the manufacturer without causing a change in the design stresses in Table 1. The manufacturer

Plywood in engineered structural components

Design is usually undertaken by a structural engineer who should have the knowledge to calculate stress distributions within composite components of timber and plywood, and within layered materials.

Designs cannot be completely undertaken using design strengths and stiffnesses, and design stresses must be specified.

When design stresses are specified for the full cross-section, the geometrical properties of the section cannot be tabulated only on the full cross-section, but must also be tabulated in terms of the lay-up for the calculation of shear stresses within the plywood. Hence the layered nature of the material has to be acknowledged at some stage in the design calculations.

If design stresses are specified on the parallel plies only, the layered nature of the material is acknowledged from the beginning and is taken into account at all stages of the calculation.

If new plywood species and combinations of species, and combinations of wood and other materials in panel form are to be developed, it is essential that more accurate methods of predicting the strength of layeredpanels should be developed. These methods will need to acknowledge the layered nature of the material and it is logical to carry this concept through to the design stage in terms of design stresses for engineered components. For sheathing applications only, design strengths and stiffnesses need to be predicted for the preparation of load-span tables.

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Table 1 (Council of Forest Industries, 1972)

Allowable Working Stresses

(Amended to provide for use of Western Softwood species other than Douglas fir as inner plies of all grades)

NOTE
These working stresses apply to Canadian fir plywood manufactured in accordance with the current edition of CSA 0121—"Douglas fir plywood". They assume normal loading and a dry service condition. "Dry service condition" means a condition in which the average equilibrium moisture content over a year is 15% or less.

Type of Stress	Allowable working stresses PSI		
	Good 2 Sides	Good 1 Side	Solid 2 Sides Solid 1 Side Sheathing Select Sheathing
Extreme fibre in bending			
Face grain parallel to span	2,065	1,890	1,770
Face grain perpendicular to span 3 plies	2,500	2,500	2,500
Face grain perpendicular to span 5 or more plies	1,665	1,665	1,665
Tension			
Parallel to face grain, 3 plies	2,430	2,220	2,080
Parallel to face grain, 5 or more plies	2,000	1,875	1,875
Perpendicular to face grain	1,665	1,665	1,665
±45° to face grain	321	303	295
Compression			
Parallel to face grain, 3 plies	1,760	1,610	1,510
Parallel to face grain, 5 or more plies	1,450	1,360	1,360
Perpendicular to face grain	1,055	1,055	1,055
± 45° to face grain	432	416	400
Bearing (on face)	440	440	440
Shear, rolling in plane of plies			
Parallel or perpendicular to face grain	56	56	56
± 45° to face grain	75	75	75
Shear through thickness			
Parallel or perpendicular to face grain	243	223	210
± 45° to face grain	493	451	425
Modulus of elasticity in bending			
Face grain parallel to span	1,800,000	1,800,000	1,800,000
Face grain perpendicular to span	1,125,000	1,125,000	1,125,000
Shearing modulus			
Face grain parallel or perpendicular to span	117,000	117,000	117,000
Face grain ± 45° to span	375,000	375,000	375,000

Table 2 (Council of Forest Industries, 1972)

Section Properties for Canadian Fir Plywood

Plywood thickness: Nominal Imperial in	No. of plies	Representative veneer thickness in inches (see note 2): Faces	Cores Perp. to Face	Centres Para. to Face	Plies parallel to face grain per 12 inch width: Net Thickness in	Area in²	Section Modulus in³	Moment of Inertia in⁴	First Moment of Area in³ (see note 3)	Plies perpendicular to face grain per 12 inch width: Net Thickness in	Area in²	Section Modulus in³	Moment of Inertia in⁴	First Moment of Area in³ (see note 3)	Approximate weight Pounds per M sq ft
1/4S	3	0.074	0.102		0.148	1.78	0.117	0.0146	0.0781	0.102	1.22	0.0208	0.00106	0	790
5/16U	3	0.097	0.097		0.194	2.33	0.163	0.0237	0.113	0.097	1.16	0.0188	0.000913	0	950
3/8S	3	0.083	0.210		0.165	1.99	0.233	0.0439	0.146	0.210	2.52	0.0882	0.00926	0	1125
3/8U	3	0.097	0.168		0.194	2.33	0.236	0.0427	0.154	0.168	2.02	0.0564	0.00474	0	1125
1/2S	5	0.061	0.126	0.126	0.248	2.98	0.292	0.0730	0.161	0.252	3.02	0.275	0.0520	0.190	1525
1/2U	5	0.099	0.099	0.099	0.297	3.56	0.388	0.0961	0.235	0.198	2.38	0.170	0.0252	0.118	1525
5/8S	5	0.061	0.168	0.168	0.290	3.48	0.390	0.122	0.207	0.336	4.03	0.489	0.123	0.339	1825
5/8U	5	0.138	0.099	0.138	0.414	4.97	0.634	0.194	0.392	0.198	2.38	0.210	0.0353	0.141	1825
11/16S	5	0.068	0.178	0.178	0.314	3.77	0.460	0.154	0.246	0.356	4.27	0.528	0.141	0.380	2050
11/16S	7	0.074	0.118	0.093	0.334	4.01	0.566	0.195	0.390	0.354	4.25	0.485	0.131	0.299	2050
3/4S	7	0.069	0.138	0.099	0.336	4.03	0.608	0.228	0.423	0.414	4.97	0.634	0.194	0.392	2225
3/4U	5	0.098	0.168	0.218	0.414	4.97	0.699	0.262	0.383	0.336	4.03	0.576	0.160	0.389	2225
3/4U	7	0.096	0.093	0.135	0.462	5.54	0.778	0.288	0.556	0.279	3.35	0.432	0.118	0.254	2225
7/8S	7	0.057	0.158	0.145	0.404	4.85	0.722	0.317	0.544	0.474	5.69	0.942	0.360	0.574	2600
7/8U	7	0.100	0.158	0.100	0.400	4.80	0.923	0.403	0.619	0.474	5.69	0.784	0.264	0.489	2600
1S	7	0.068	0.198	0.135	0.406	4.87	0.900	0.450	0.650	0.594	7.13	1.27	0.550	0.791	3000
1S	9	0.076	0.095	0.156	0.620	7.44	1.28	0.638	0.891	0.380	4.56	0.855	0.362	0.572	3000
1U	7	0.095	0.155	0.155	0.500	6.00	1.10	0.530	0.784	0.465	5.58	0.951	0.369	0.577	3000
1U	9	0.117	0.117	0.095	0.519	6.23	1.30	0.640	0.852	0.468	5.62	0.855	0.322	0.595	3000
1 1/8S	7	0.068	0.198	0.198	0.532	6.38	1.17	0.659	0.902	0.594	7.13	1.55	0.768	0.941	3375
1 1/8S	9	0.087	0.121	0.156	0.642	7.70	1.53	0.864	1.06	0.484	5.81	1.18	0.564	0.804	3375
1 1/8U	7	0.095	0.198	0.155	0.500	6.00	1.27	0.694	0.898	0.594	7.13	1.36	0.615	0.839	3375
1 1/8U	9	0.095	0.155	0.095	0.475	5.70	1.31	0.717	0.855	0.620	7.44	1.32	0.596	0.930	3375
1 1/4S	9	0.058	0.162	0.162	0.602	7.22	1.46	0.916	1.04	0.648	7.78	1.83	1.04	1.26	3750
1 1/4S	11	0.088	0.130	0.106	0.600	7.20	1.72	1.07	1.21	0.650	7.80	1.64	0.880	1.10	3750
1 1/4U	9	0.120	0.120	0.178	0.774	9.29	2.11	1.33	1.45	0.480	5.76	1.28	0.646	0.858	3750

S indicates sanded panel. U indicates unsanded panel.

NOTES

1) Plywood panel lay-up varies slightly among manufacturers. In each case the table shows the most conservative figure for plies parallel to face grain.
2) Table shows representative veneer thicknesses. In practice, actual veneer thicknesses may vary slightly from those shown. Veneers must be measured if precise values are desired.
3) First moment (statical moment) of area about the neutral axis of all material lying outside (above or below) the critical rolling shear plane excluding the plies perpendicular to the span.

Table 3 (British Standards Institution, 1967)

CP 112 : 1967

DRY GRADE STRESSES AND MODULI FOR FINNISH EUROPEAN BIRCH PLYWOOD (FINPLY-EXTERIOR)

Type and direction of stress and modulus	Value of stress or modulus
	lbf/in^2
Extreme fibre in bending	
Face grain parallel to span, 5 ply	2 300
Face grain parallel to span, 7 or more ply	2 000
Face grain perpendicular to span, 5 and 7 ply	1 220
Face grain perpendicular to span, 9 or more ply	1 570
Face grain 45° to span	1 240
Tension	
Parallel to face grain, 5 and 7 ply	2 100
Parallel to face grain, 9 or more ply	1 890
Perpendicular to face grain	1 390
45° to face grain	670
Compression	
Parallel to face grain, 5 and 7 ply	1 180
Parallel to face grain, 9 or more ply	1 040
Perpendicular to face grain	780
45° to face grain	710
Bearing	
On face	500
Shear, rolling in plane of plies	
Parallel and perpendicular to face grain	125
45° to face grain	125
Panel shear	
Parallel and perpendicular to face grain	450
45° to face grain	1 050
Modulus of elasticity in bending	
Face grain parallel to span, 5 ply	1 470 000
Face grain parallel to span, 7 or more ply	1 210 000
Face grain perpendicular to span, 5 and 7 ply	600 000
Face grain perpendicular to span, 9 or more ply	812 000
Face grain 45° to span	296 000
Modulus of elasticity in tension and compression	
Parallel to face grain	1 260 000
Perpendicular to face grain, 5 and 7 ply	1 030 000
Perpendicular to face grain, 9 or more ply	1 100 000
45° to face grain	316 000
Modulus of rigidity	
Parallel and perpendicular to face grain	116 000
45° to face grain	330 000

Table 4 (British Standards Institution, 1967)

CP 112 : 1967

DIMENSIONS AND PROPERTIES OF FINNISH EUROPEAN BIRCH PLYWOOD (FINPLY-EXTERIOR)

Surface condition	Thickness		Number of plies	Section properties for a 12-inch width (Note 1)			Weight per 1000 ft²
				Area	Section modulus	Second moment of area	
	mm	in		in²	in³	in⁴	lb
Sanded	6·5	0·256	5	3·07	0·131	0·0168	930
	9·3	0·366	7	4·39	0·268	0·0490	1 330
	12·0	0·472	9	5·66	0·446	0·105	1 720
	14·8	0·583	11	7·00	0·680	0·198	2 120
	17·6	0·693	13	3·32	0·960	0·333	2 520
	20·4	0·803	15	9·64	1·29	0·518	2 920
	23·2	0·913	17	10·96	1·67	0·761	3 330
Unsanded	7·0	0·276	5	3·31	0·152	0·0210	1 020
	9·8	0·386	7	4·63	0·298	0·0575	1 410
	12·6	0·496	9	5·95	0·492	0·122	1 810
	15·4	0·606	11	7·27	0·734	0·223	2 210
	18·2	0·717	13	8·60	1·03	0·369	2 620
	21·0	0·827	15	9·92	1·37	0·566	3 020
	23·8	0·937	17	11·24	1·76	0·823	3 430

NOTE 1. In all applications for unsanded plywood where the direction of the face veneer is across the span, the values for section modulus and moments of inertia shown for sanded plywood should be used.

Table 5 (Vermeyden, 1967)

F, W and I for Finnish birch plywood, for a strip 1 cm wide

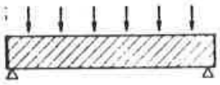
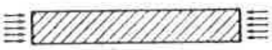
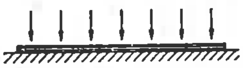
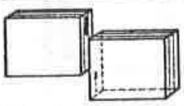
***F*, *W* en *I* van Fins berketriplex, voor een strook van 1 cm breed**

t in mm	6	9	12	15	18	21	24
F in cm^2	0,6	0,9	1,2	1,5	1,8	2,1	2,4
W in cm^3	0,060	0,135	0,240	0,375	0,540	0,735	0,960
I in cm^4	0,0180	0,0608	0,144	0,281	0,486	0,772	1,152

Table 6 (Vermeyden, 1967)

Permissible stresses and moduli of elasticity for Finnish birch plywood (BB, WG) in kgf/cm^2, moisture content < 21%

Toelaatbare spanningen en elastische grootheden van Fins berken constructietriplex (kwaliteit BB of WG), in kgf/cm^2, geldend voor een vochtgehalte < 21%.

			hoek tussen belastings- of overspanningsrichting en vezelrichting deklaag				
			0° (//)		90° (⊥)		45°
			t = 6 en 9 mm	$t \geqq$ 12 mm	t = 6 en 9 mm	$t \geqq$ 12 mm	
buiging ⊥ plaatvlak bending ⊥ panel		$\bar{\sigma}_{b1}$	150	150	90	110	85
buiging // plaatvlak bending // panel		$\bar{\sigma}_{b2}$	$\bar{\sigma}_t$ resp. $\bar{\sigma}_{d1}$ aanhouden in trek- resp. drukzone *				
trek tension		$\bar{\sigma}_t$	150	135	75	75	40
druk // plaatvlak compression // panel		$\bar{\sigma}_{d1}$	80	70	55	55	50
druk ⊥ plaatvlak compression ⊥ panel		$\bar{\sigma}_{d2}$	30	30	30	30	30
rolafschuiving rolling shear		$\bar{\tau}_r$	6	6	6	6	6
paneelafschuiving panel shear		$\bar{\tau}_p$	30	30	30	30	75
elasticiteitsmodulus elastic moduli							
bij buiging ⊥ plaatvlak bending ⊥ panel		E_{b1}	85 000	85 000	42 000	55 000	20 000
bij buiging // plaatvlak bending // panel		E_{b2}	85 000	85 000	50 000	55 000	20 000
bij trek en druk tension and compression		E_t, E_d	85 000	85 000	50 000	55 000	20 000
glijdingsmodulus rigidity modulus		G	8 000	8 000	8 000	8 000	24 000

* amended by Vermeyden (1969) to read "$\bar{\sigma}_t$ to be used in tension and compression zone"

Table 7 (Deutsche Normen, 1969)

DIN 1052

9.2. Furnierplatten

9.2.1. Für tragende Bauteile dürfen ohne weitere Eignungsnachweise Furnierplatten nach DIN 68 705 Blatt 3, verwendet werden.

9.2.2. Für Furnierplatten nach Abschnitt 9.2.1 sind die Spannungen nach Tabelle 8 zulässig. Im Lastfall HZ (siehe Abschnitt 4.1.2) können die zulässigen Spannungen um 15 % erhöht werden.

9.2.3. Die zulässigen Spannungen für Zug und Druck in Plattenebene unter $30° \leqq \alpha \leqq 60°$ betragen 20 kp/cm². Dabei ist α der Winkel zwischen der Kraft- und der Faserrichtung der Deckfurniere. Für $0° \leqq \alpha \leqq 30°$ darf zwischen 80 kp/cm² $\geqq$ zul $\sigma_{Z,D} \geqq$ 20 kp/cm² und für $60° \leqq \alpha \leqq 90°$ zwischen 20 kp/cm² $\leqq$ zul $\sigma_{Z,D} \leqq$ 40 kp/cm² geradlinig interpoliert werden.

Tabelle 8. Zulässige Spannungen im Lastfall H für Furnierplatten nach DIN 68 705 Blatt 3, bezogen auf den Vollquerschnitt

Zeile	Art der Beanspruchung	Zulässige Spannungen parallel der Faserrichtung der Deckfurniere kp/cm²	Zulässige Spannungen rechtwinklig zur Faserrichtung der Deckfurniere kp/cm²
1	Biegung rechtwinklig zur Plattenebene zul σ_B	130	50
2	Biegung in Plattenebene zul σ_B	90	60
3	Zug in Plattenebene zul σ_Z	80	40
4	Druck in Plattenebene zul σ_D	80	40
5	Druck rechtwinklig zur Plattenebene zul σ_D	30	
6	Abscheren in Plattenebene zul τ	9	
7	Abscheren rechtwinklig zur Plattenebene zul τ	18	

9.2 Plywood boards

9.2.1 Plywood boards to DIN 68705, Sheet 3 may be used for stressed construction members without any further proof of suitability.

9.2.2 The stresses in Table 8 are allowable for plywood boards under Section 9.2.1. Under loading condition HZ (See Section 4.1.2) the permissible stresses may be increased by 15%.

9.2.3 The permissible tensile and compressive stresses in the plane of the board under $30° \leqq \alpha \leqq 60°$ are 20 kp/cm², where α is the angle between the force and the direction of grain of the top ply. For $0° \leqq \alpha \leqq 30°$ one may interpolate linearly between 80 kp/cm² $\geqq$ zul $\sigma_{Z,D} \geqq$ 20 kp/cm² and for $60° \leqq \alpha \leqq 90°$ one may interpolate linearly between 20 kp/cm² $\leqq$ zul $\sigma_{Z,D} \leqq$ 40 kp/cm²

Table 8 Permissible stresses under loading condition H for plywood boards to DIN 68705, Sheet 3, relative to the whole cross section

Line	Type of stress	Permissible stresses parallel to the grain of of the top ply kp/cm²	Permissible stresses Normal to the grain of top ply kp/cm²
1	Bending normal to the plane of the board zul σ_B	130	50
2	Bending in the plane of the board zul σ_B	90	60
3	Tension in the plane of the board zul σ_Z	80	40
4	Compression in the plane of the board zul σ_D	80	40
5	Compression normal to the plane of the board zul σ_D	30	
6	Shear in the plane of the board zul τ	9	
7	Shear normal to the plane of the board zul τ	18	

Table 8 (Noren, 1964)

SUGGESTED WORKING STRESSES FOR SWEDISH PINE PLYWOOD OF STANDARD CONSTRUCTION AND STANDARD STRUCTURAL QUALITY

Symbols

A	Plywood panel with one good side, grade A/X, B/X or BB/X
B	Plywood panel with two good sides, grade B or BB
I	Parallel to the fibre direction of face veneers
II	At right angles to the fibre direction of face veneers

Table A *Thickness of plywood and cross section data (sanded panels)*

Thickness t mm	Number of plies	Area quotient** A_I:A_{II}	Second moment quotient** I_I:I_{II}
4	3	61:39	94:6
6	3	41:59	79:21
7	5	56:44	72:28
8	5	61:39	68:32
10	5	49:51	57:43
12	7	61:39	64:36
15	7	49:51	52:48
18	9	42:58	50:50

** The quotients should be used in determining deformations (see table F).

Table B *Working stresses and moments in bending*

Thickness t mm	Number of plies	Bending stress *k* lb. per sq. in. A I	A II	B I	B II	Moment *M* lb. per in. width A I	A II	B I	B II
4	3	2130	430	2560	430	8.8	1.8	10.6	1.8
6	3	1710	850	2130	850	15.9	7.9	19.8	7.9
7	5	1710	1140	2130	1140	21.6	14.3	27	14.3
8	5	1710	1140	2130	1140	28	18.8	35	18.8
10	5	1420	1420	1710	1420	37	37	44	37
12	7	1710	1140	2130	1140	64	42	79	42
15	7	1420	1420	1710	1420	84	84	99	84
18	9	1420	1420	1710	1420	119	119	143	119

Table C *Working stresses and forces in tension*

Thickness t mm	Number of plies	Tensile stress σ_s lb. per sq. in. A I	A II	B I	B II	Tensile force *D* lb. per in. width A I	A II	B I	B II
4	3	1280	850	1560	850	230	155	285	155
6	3	1000	1280	1140	1280	270	350	310	350
7	5	1280	1140	1420	1140	410	360	450	360
8	5	1280	1000	1560	1000	465	360	570	360
10	5	1000	1280	1280	1280	450	580	580	580
12	7	1420	1000	1560	1000	780	540	780	540
15	7	1140	1280	1280	1280	900	870	870	870
18	9	1140	1420	1280	1420	930	1160	1050	1160

Table D *Working stresses and forces in compression*

Thickness t mm	Number of plies	Compressive stress σ_t lb. per sq. in. A I	A II	B I	B II	Compressive force *T* lb. per in. width A I	A II	B I	B II
4	3	1000	710	1140	710	180	130	205	130
6	3	710	1070	780	1070	195	290	215	290
7	5	920	850	1000	850	290	270	315	270
8	5	1070	710	1140	710	390	260	415	260
10	5	780	920	920	920	355	420	420	420
12	7	1070	710	1140	710	580	390	620	390
15	7	850	920	920	920	580	630	630	630
18	9	850	1070	850	1070	700	870	700	870

Table E *Working stresses in shear*

Type of shear	Angle of force direction to fibre direction 0°	45°	90°
Panel shear			
τ_p, lb. per sq. in.	350	850	350
Shear in plane of plies			
τ_r, lb. per sq. in.	115	140	70
At panel edge*	85	115	60

*Closer to the edge than 5 × t or two inches.

Table F *Elasticity moduli to be applied to total cross section (based on $E_1 = 1.7 \cdot 10^6$ lb. per sq. in. and $E_2 = 0$)*

Type of stress	Modulus of elasticity, lb. per sq. in. E_I	E_{II}
Bending	$1.7 \cdot 10^6 \frac{I_I}{I}$	$1.7 \cdot 10^6 \frac{I_{II}}{I}$
Tension, compression	$1.7 \cdot 10^6 \frac{A_I}{A}$	$1.7 \cdot 10^6 \frac{A_{II}}{A}$

The cross-sectional fractions are found in table I. Note that

$$I_I/I_{II} = 100\frac{I_I}{I} : 100\frac{I_{II}}{I} \text{ and } A_I : A_{II} = 100\frac{A_I}{A} : 100\frac{A_{II}}{A}.$$

Table G *Moduli of rigidity in panel shear (shear through thickness)*

Angle of shear stress to fibre direction	0°, 90°	45°
Modulus of rigidity, lb. per sq. in.	$0.1 \cdot 10^6$	$0.43 \cdot 10^6$**

**Panel composed of at least five plies.

Table 9 (Larsen, 1971)

Finply Exterior Plywood WBP

Pudset. Nominelle værdier. Belastningsgruppe A. Fugtklasse I. Normale belastningstilfælde.

Sanded. Characteristic values. Long term. Dry

Tyk-kelse	Bøjning						Træk i skiveplanen			Træk og tryk i skiveplanen			Tryk i skiveplanen			Tryk vinkelret på skiveplan	Forskydning				
	Styrke i kpcm/cm			Stivhed i kpcm²/cm			Styrke i kp/cm			Stivhed i kp/cm			Styrke i kp/cm			Styrke i kp/cm²	Styrke i kp/cm²			Stivhed i kp/cm²	
																	gennem tykkelsen		mellem lag		
mm	m_0	m_{45}	m_{90}	$(EI)_0$	$(EI)_{45}$	$(EI)_{90}$	n_0	n_{45}	n_{90}	$(Ed)_0$	$(Ed)_{45}$	$(Ed)_{90}$	n'_0	n'_{45}	n'_{90}	s'_{90}	t_0	t_{45}	t_{90}	G_0	G_{45}
6,5	18	9	8	2.430	500	960	141	48	95	59.000	14.000	48.000	87	50	59						
9,3	33	19	23	5.900	1.470	3.620	202	67	137	84.000	20.000	69.000	123	72	111						
12,0	55	32	42	12.700	3.170	8.780	235	87	176	109.000	26.000	95.000	143	93	143						
14,8	84	49	64	23.800	5.940	16.500	290	108	218	134.000	32.000	117.000	176	113	176						
17,6	119	69	90	40.000	9.990	27.600	344	129	259	159.000	38.000	139.000	210	136	210						
20,4	160	92	121	62.200	15.500	43.100	400	148	300	185.000	44.000	161.000	242	157	242						
23,2	207	119	157	91.500	22.900	63.500	455	169	342	210.000	50.000	183.000	276	179	276	56	50	105	14	8.300	25.000
26,0	260	150	197	129.000	32.200	89.400	510	189	382	235.000	56.000	205.000	310	200	310						
28,8	319	183	242	175.000	43.900	122.000	565	210	424	261.000	62.000	228.000	343	221	343						
31,6	384	221	291	232.000	57.900	161.000	620	231	465	286.000	68.000	250.000	376	244	377						
34,4	456	262	344	298.000	74.500	207.000	673	250	506	311.000	74.000	272.000	410	264	410						
37,2	523	307	403	378.000	94.300	262.000	728	272	548	337.000	80.000	294.000	443	287	444						
40,0	615	356	467	469.000	117.000	325.000	785	291	588	362.000	86.000	316.000	476	308	476						
42,8	705	406	533	575.000	144.000	398.000	838	312	630	387.000	92.000	338.000	510	330	510						

Thickness	Bending Strength	Stiffness	Tension // panel Strength	Tension and compression // panel Stiffness	Compression // panel Strength	Comp ⊥ panel Strength	Shear strength Panel Rolling	Stiffness

Table 10 (Booth and Reece, 1967)

Table **Grade stresses for Canadian Douglas fir plywood**

18 per cent moisture content
Long term loading
N.B. Parallel plies only approach

Type of stress	*Good 2 sides*	*Good 1 side*	*Solid 2 sides, Solid 1 side, or Unsanded sheathing*
		lbf/in^2	
Extreme fibre in bending			
Face grain parallel to span	1740	1600	1500
Face grain perpendicular to span 3 plies	2110	2110	2110
Face grain perpendicular to span 5 or more plies	1410	1410	1410
Tension			
Parallel to face grain, 3 ply	2 050	1 880	1 760
Parallel to face grain, 5 or more ply	1 690	1 580	1 580
Perpendicular to face grain	1 410	1 410	1 410
45° to face grain	270	260	250
Compression			
Parallel to face grain, 3 ply	1 430	1 310	1 230
Parallel to face grain, 5 or more ply	1 180	1 110	1 110
Perpendicular to face grain	860	860	860
45° to face grain	350	340	330
Bearing (on face)	380	380	380
Rolling shear			
Parallel and perpendicular to face grain	50	50	50
45° to face grain	65	65	65
Panel shear			
Parallel and perpendicular to face grain	210	190	180
45° to face grain	420	390	360
Modulus of elasticity in bending			
Face grain parallel to span	1 750 000	1 750 000	1 750 000
Face grain perpendicular to span	1 100 000	1 100 000	1 100 000
Modulus of rigidity			
Parallel and perpendicular to face grain	112 000	112 000	112 000
45° to face grain	358 000	358 000	358 000

Table 11 (Plywood Manufacturers Association of British Columbia, 1961)

MOMENTS OF INERTIA, SECTION MODULI, VENEER THICKNESSES AND AREAS FOR SELECTED PLYWOOD CONSTRUCTIONS

Plywood Thickness (Nominal) Inches	No. of Plies	Veneer Thickness (Nominal) Inches			Plies Parallel to Face Grain Only 12 inch widths				Plies Perpendicular to Face Grain Only — 12 inch widths				Weight lbs. per M. sq. ft. (Approx)
		Faces†	Cores (Perp. to Face)	Centres (Para. to Face)	Thickness (Net) Inches	Area $inches^2$	Section Modulus $inches^3$	Moment of Inertia $inches^4$	Thickness (Net) inches	Area $inches^2$	Section Modulus $inches^3$	Moment of Inertia $inches^4$	
1/4 S*	3	1/10	1/10		0.148	1.78	0.117	0.0146	0.102	1.22	0.0208	0.00106	790
5/16U*	3	1/10	1/10		0.194	2.33	0.163	0.0237	0.097	1.16	0.0188	0.000913	950
3/8 S	3	1/10	1/5		0.165	1.98	0.232	0.0435	0.210	2.52	0.0882	0.00926	1125
3/8 U	3	1/10	1/6		0.194	2.33	0.236	0.0427	0.168	2.02	0.0564	0.00474	1125
1/2 S	5	1/10	2 @ 1/8	1/8	0.248	2.98	0.292	0.0730	0.252	3.02	0.275	0.0520	1525
1/2 U	5	1/10	2 @ 1/10	1/10	0.297	3.56	0.389	0.0961	0.198	2.38	0.170	0.0252	1525
5/8 S	5	1/10	2 @ 1/6	1/6	0.289	3.47	0.388	0.121	0.336	4.03	0.488	0.123	1825
5/8 U	5	1/7	2 @ 1/10	1/7	0.414	4.97	0.634	0.194	0.198	2.38	0.210	0.0353	1825
3/4 S	7	1/10	3 @ 1/7	2 @ 1/10	0.336	4.03	0.608	0.228	0.414	4.97	0.634	0.194	2225
3/4 U	5	1/10	2 @ 1/6	7/32	0.413	4.96	0.695	0.260	0.336	4.03	0.578	0.160	2225
3/4 U	7	1/10	3 @ 1/10	2 @ 1/8	0.462	5.55	0.780	0.289	0.279	3.35	0.432	0.119	2225
7/8 S	7	1/10	3 @ 1/6	2 @ 1/7	0.403	4.82	0.711	0.311	0.473	5.68	0.940	0.359	2600
7/8 U	7	1/10	3 @ 1/6	2 @ 1/10	0.400	4.80	0.922	0.402	0.473	5.68	0.782	0.263	2600
1 S	7	1/10	3 @ 3/16	2 @ 1/7	0.406	4.87	0.900	0.450	0.594	7.13	1.275	0.550	3000
1 S	9	1/10	4 @ 1/10	3 @ 1/6	0.620	7.44	1.280	0.640	0.380	4.56	0.850	0.360	3000
1 U	7	1/10	3 @ 1/6	2 @ 1/6	0.500	6.00	1.098	0.530	0.465	5.58	0.953	0.369	3000
1 U	9	1/8	4 @ 1/8	3 @ 1/10	0.519	6.23	1.297	0.640	0.468	5.62	0.853	0.321	3000
1-1/8 S	7	1/10	3 @ 3/16	2 @ 3/16	0.531	6.37	1.163	0.654	0.594	7.13	1.558	0.770	3375
1-1/8 S	9	1/10	4 @ 1/8	3 @ 1/6	0.641	7.69	1.490	0.838	0.484	5.81	1.209	0.586	3375
1-1/8 U	7	1/10	3 @ 3/16	2 @ 1/6	0.500	6.00	1.267	0.693	0.594	7.13	1.362	0.616	3375
1-1/8 U	9	1/10	4 @ 1/6	3 @ 1/10	0.475	5.70	1.309	0.717	0.620	7.44	1.318	0.596	3375
1-1/4 S	9	1/10	4 @ 1/6	3 @ 1/6	0.602	7.22	1.466	0.916	0.648	7.78	1.829	1.037	3750
1-1/4 S	11	1/7	5 @ 1/7	4 @ 1/10	0.600	7.20	1.718	1.074	0.650	7.80	1.638	0.880	3750
1-1/4 U	9	1/8	4 @ 1/8	3 @ 3/16	0.774	9.29	2.114	1.326	0.480	5.76	1.274	0.646	3750

* S means Sanded, U means Unsanded.
† For Sanded panels, thickness is before sanding.

TABLE 12 DRY GRADE STRESSES AND MODULI FOR CANADIAN DOUGLAS FIR PLYWOOD (SHEATHING GRADE)

(Stresses and moduli expressed in lbf/in²)

Type and direction of stress and modulus	Grade stresses and moduli for the following nominal thicknesses in inches (with the total number of plies in parenthesis)													
	1/4 (3)	5/16 (3)	3/8 (3)	1/2 (5)	5/8 (5)	3/4 (7)	3/4 (5)	7/8 (7)	1 (7)	1 (9)	1 1/8 (7)	1 1/8 (9)	1 1/4 (9)	1 1/4 (11)
EXTREME FIBRE IN BENDING														
Face grain parallel to span	—	1 440	1 350	1 190	1 270	1 060	930	900	880	1 000	790	820	1 010	—
Face grain perpendicular to span	—	240	460	480	400	560	730	720	720	620	810	780	570	—
TENSION														
Parallel to face grain	—	1 180	940	950	1 070	990	870	720	820	830	720	690	980	—
Perpendicular to face grain	—	470	660	570	460	530	630	760	680	670	770	800	640	—
45° to face grain	250	250	250	250	250	250	250	250	250	250	250	250	250	250
COMPRESSION														
Parallel to face grain	—	820	660	660	750	690	610	510	580	580	510	480	690	—
Perpendicular to face grain	—	290	400	350	280	320	390	470	410	410	470	490	330	—
45° to face grain	330	330	330	330	330	330	330	330	330	330	330	330	330	330
BEARING														
On face	380	380	380	380	380	380	380	380	380	380	380	380	380	380
SHEAR, ROLLING IN PLANE OF PLIES														
Parallel and perpendicular to face grain	50	50	50	50	50	50	50	50	50	50	50	50	50	50
45° to face grain	70	70	70	70	70	70	70	70	70	70	70	70	70	70
PANEL SHEAR														
Parallel and perpendicular to face grain	180	180	180	180	180	180	180	180	180	180	180	180	180	180
45° to face grain	360	360	360	360	360	360	360	360	360	360	360	360	360	360
MODULUS OF ELASTICITY IN BENDING														
Parallel to face grain (10^3 lbf/in²)	—	1 690	1 580	1 400	1 500	1 270	1 120	1 090	1 070	1 200	970	1 000	1 210	—
Perpendicular to face grain (10^3 lbf/in²)	—	90	160	270	220	360	450	470	480	400	550	530	400	—
MODULUS OF ELASTICITY IN TENSION AND COMPRESSION														
Parallel to face grain (10^3 lbf/in²)	—	1 200	980	1 080	1 210	1 130	1 010	850	950	960	850	810	1 110	—
Perpendicular to face grain (10^3 lbf/in²)	—	400	540	470	390	450	520	620	560	550	620	650	460	—
MODULUS OF RIGIDITY														
Parallel and perpendicular to face grain (10^3 lbf/in²)	110	110	110	110	110	110	110	110	110	110	110	110	110	110
45° to face grain (10^3 lbf/in²)	360	360	360	360	360	360	360	360	360	360	360	360	360	360

Table 13 (British Standards Instituion, 1967)

CP 112 : 1967 DIMENSIONS AND PROPERTIES OF CANADIAN DOUGLAS FIR PLYWOOD

Nominal thickness	Surface condition (Note 1)	Total number of plies	Nominal veneer thicknesses in inches (with numbers of plies in parenthesis)			Section properties for a 12-inch width				Weight per 1000 ft^2
			Two faces	Parallel to face	Perpendicular to face	Thickness (net)	Area	Section modulus	Second moment of area	
in						in	in^2	in^3	in^4	lb
5/16	U	3	1/10		1/10(1)	0·291	3·49	0·169	0·0246	950
3/8	U	3	1/10		1/6 (1)	0·362	4·35	0·262	0·0474	1 125
1/2	U	5	1/10	1/10(1)	1/10(2)	0·495	5·94	0·490	0·1213	1 525
5/8	U	5	1/7	1/7 (1)	1/10(2)	0·612	7·35	0·749	0·229	1 825
3/4	U	5	1/10	7/32(1)	1/6 (2)	0·749	8·99	1·121	0·420	2 225
3/4	U	7	1/10	1/8 (2)	1/10(3)	0·741	8·90	1·101	0·408	2 225
7/8	U	7	1/10	1/10(2)	1/6 (3)	0·873	10·48	1·523	0·665	2 600
1	U	7	1/10	1/6 (3)	1/6 (3)	0·965	11·58	1·863	0·899	3 000
1	U	9	1/8	1/10(3)	1/6 (4)	0·987	11·85	1·947	0·961	3 000
1 1/8	U	7	1/10	1/6 (2)	3/10(3)	1·094	13·13	2·393	1·309	3 375
1 1/8	U	9	1/10	1/10(3)	1/6 (4)	1·095	13·14	2·398	1·313	3 375
1 1/4	U	9	1/8	3/10(3)	1/8 (4)	1·254	15·05	3·145	1·972	3 750

NOTE 1 U = Unsanded.

Table 14 (Vermeyden, 1969)

Permissible stresses and elastic moduli of Canadian Oregon pine plywood; unsanded; valid for moisture content < 21%; $\bar{\sigma}$ and $\bar{\tau}$ in kgf/cm^2; E and G in 1000 kgf/cm^2

Toelaatbare spanningen en elasticiteitsmodulussen van Canadees Oregon-pinetriplex; *ongeschuurd;* geldend voor een vochtgehalte < 21%. $\bar{\sigma}$ en $\bar{\tau}$ in kgf/cm^2; $\bar{E}$ en $\bar{G}$ in 1 000 kgf/cm^2

	t in mm			8	10	13	16	19	19	22	25	25	28	28	32
	(inch)			5/16	3/8	1/2	5/8	3/4	3/4	7/8	1	1	1 1/8	1 1/8	1 1/4
number of plies	aantal lagen			3	3	5	5	7	5	7	7	9	7	9	9
bending ⊥ panel	buiging ⊥ plaatvlak	$\bar{\sigma}_{b1}$	//[1])	80	80	80	80	70	70	70	60	70	60	60	70
			⊥	10	20	30	25	40	50	50	50	40	60	50	40
bending // panel	buiging // plaatvlak	$\bar{\sigma}_{b2}$	//	50	50	50	70	70	60	50	60	60	50	50	70
			⊥	20	25	25	30	35	45	50	50	50	50	60	40
tension	trek	$\bar{\sigma}_t$	//	40	40	50	70	70	60	50	60	60	50	50	70
			⊥	10	20	20	30	35	45	50	50	50	50	60	40
			45°	10	10	15	15	15	20	20	20	20	20	20	20
compression // panel	druk // plaatvlak	$\bar{\sigma}_{d1}$	//	50	45	45	50	50	45	40	40	40	35	35	50
			⊥	20	25	25	20	20	25	30	30	30	30	35	25
			45°	20	20	25	25	25	25	25	25	25	25	25	25
compression ⊥ panel	druk ⊥ plaatvlak	$\bar{\sigma}_{d2}$		25	25	25	25	25	25	25	25	25	25	25	25
rolling shear	rolafschuiving	$\bar{\tau}_r$		3,5	3,5	3,5	3,5	3,5	3,5	3,5	3,5	3,5	3,5	3,5	3,5
panel shear	paneelafschuiving	$\bar{\tau}_p$	//, ⊥	15	15	15	15	15	15	15	15	15	15	15	15
			45°	25	25	25	25	25	25	25	25	25	25	25	25
elastic modulus bending ⊥ panel	elasticiteitsmodulus bij buiging ⊥ plaatvlak	$\bar{E}_{b1}$	//	80	80	80	80	80	80	80	70	80	70	70	80
			⊥	5	5	25	20	25	30	30	30	30	40	40	30
bending // panel tension and compression	bij buiging // plaatvlak bij trek en druk	$\bar{E}_{b2}$, $\bar{E}_t$, $\bar{E}_d$	//	80	65	70	80	80	70	60	65	80	60	60	80
			⊥	25	30	30	30	30	35	45	40	30	45	45	30
rigidity modulus	glijdingsmodulus	$\bar{G}$	//, ⊥	7,5	7,5	7,5	7,5	7,5	7,5	7,5	7,5	7,5	7,5	7,5	7,5
			45°	25	25	25	25	25	25	25	25	25	25	25	25

[1]) belastings- of overspanningsrichting t.o.v. vezelrichting deklaag.
load or span direction with respect to direction of face grain

Table 15 (Nordic Building Regulations Committee, 1973)

Douglas Fir Plywood
Exterior sheating (unsanded)

Nominel Values. Long time loading. Interior.
Characteristic strength values = 1.3 x nominel values.

Thickness		Bending						Tension in plane		
		Strength in kpcm/cm			Stiffness in $kpcm^2/cm$			Strength in kp/cm		
inch[1])	mm	m_0	m_{45}	m_{90}	$(EI)_0$	$(EI)_{45}$	$(EI)_{90}$	n_0	n_{45}	n_{90}
3/8"	9,2	18		6	7.100		500	85	23	59
1/2"	12,6	30		13	16.100		2.650	118	31	70
5/8"	15,6	50		16	32.400		3.720	165	38	70
3/4" (5)	19,0	55		43	44.600		16.800	164	47	118
3/4" (7)	18,8	61		32	48.500		12.400	184	46	98
7/8"	22,2	73		58	68.600		27.800	158	54	167
1" (7)	24,5	86		71	90.500		38.800	199	60	164
1" (9)	25,1	102		64	108.000		33.900	206	62	165
1 1/8" (7)	27,8	100		101	119.000		64.700	199	68	208
1 1/8" (9)	27,8	103		98	123.000		62.700	189	68	218
1 1/4"	31,9	166		95	224.000		68.000	308	78	169

Thickness	Tension and compression			Compression in plane			Comp. perp.	Shear				
								Strength in kp/cm^2				
	Stiffness in kp/cm			Strength in kp/cm			kp/cm^2	Through thickn.		Roll.	Stiffness in kp/cm^2	
inch[1])	$(Ed)_0$	$(Ed)_{45}$	$(Ed)_{90}$	n'_0	n'_{45}	n'_{90}	s'_{90}	t_0	t_{45}	t_{90}	G_0	G_{45}
3/8"	57.000		33.000	60	31	37						
1/2"	93.000		38.800	82	40	43						
5/8"	130.000		38.800	114	50	43						
3/4" (5)	130.000		65.800	113	61	72						
3/4" (7)	145.000		54.600	127	60	60						
7/8"	125.000		92.800	110	71	102						
1" (7)	157.000		91.000	137	78	100	40	20	40	4	7.900	25.000
1" (9)	163.000		91.500	143	80	100						
1 1/8" (7)	157.000		116.000	137	89	127						
1 1/8" (9)	149.000		121.000	130	89	133						
1 1/4"	242.000		94.000	212	102	103						

1) (5), (7) and (9) denotes number of plies

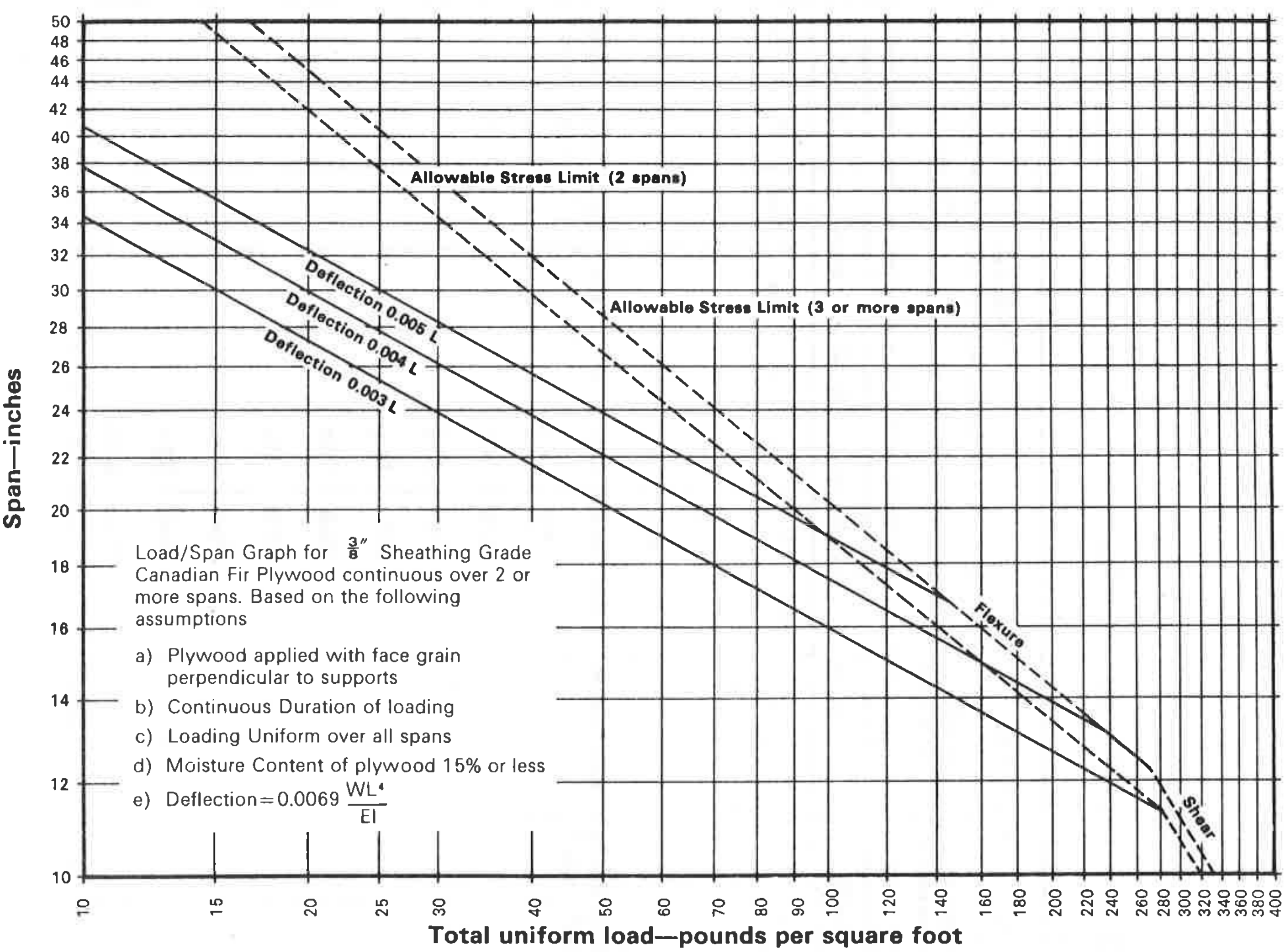
TABLE 16 DOUGLAS FIR PLYWOOD : SHEATHING GRADE

LONG TERM LOADING
18% MOISTURE CONTENT

THICKNESS	BENDING				DIRECT FORCE								SHEAR							BEARING
					TENSION IN PLANE			TENSION + COMPRESSION		COMPRESSION IN PLANE			ROLLING			PANEL				
	STRENGTH lbf.in/ft		STIFFNESS $lbf.in^2/ft$		STRENGTH lbf/ft			STIFFNESS lbf/ft		STRENGTH lbf/ft			STRENGTH lbf/ft		STRESS lbf/in^2	STRESS lbf/in^2		STIFFNESS lbf/in^2		STRESS lbf/in^2
in*	m_0	m_{90}	$(EI)_0$	$(EI)_{90}$	n_0	n_{45}	n_{90}	0	90	n'_0	n'_{45}	n'_{90}	v_0	v_{90}	τ_r	τ_{p0}	τ_{p45}	G_0	G_{45}	
5/16													126	–						
3/8													166	–						
1/2	580	240	168×10^3	28×10^3	5620	1490	3360	6230×10^3	2620×10^3	3950	1960	2050	245	128	50	180	360	112×10^3	358×10^3	380
5/8													297	150						
3/4 (5)													410	247						
3/4 (7)													311	279						
7/8													391	324						
1 (7)													406	384						
1 (9)													450	325						
1 1/8 (7)													464	440						
1 1/8 (9)													503	384						
1 1/4													550	452						

* (5), (7), and (9) denotes number of plies

Graph 1 (Plywood Manufacturers of British Columbia , 1967)



The graphs enable the correct thickness of plywood to be selected for a particular load over spans ranging from 10" centres to 50" centres. They apply only to PMBC EXTERIOR Sheathing Grade fir plywood of thicknesses $\frac{3}{8}$", $\frac{1}{2}$", $\frac{5}{8}$", $\frac{3}{4}$" (5 ply) and $\frac{3}{4}$" (7 ply).
The graphs are intended for storage loads only and are therefore based on continuous duration of loading uniformly distributed over all spans. No allowance has been made for point loads.
Two flexure lines are shown on each graph, one for the two span condition and the other for three or more spans. The flexure lines represent those points where the load and span combination results in the bending stresses being equal to the allowable stress limits of the plywood. The shear lines represent load and span combinations where rolling shear is critical for both two, and three or more, spans. Loading the plywood above the allowable stress limit line may result in failure. The deflection lines indicate appearance limits only. A deflection of 0.003 (1/333) of the span is very slight. The 0.004 (1/250) of the span deflection is barely visible to the eye. A deflection of 0.005 (1/200) of the span will appear as a slight sag. The actual conditions of each application will determine the permissible deflection.

APPENDIX to

THE PRESENTATION OF STRUCTURAL DESIGN DATA FOR PLYWOOD

Comparative outline calculations for

1 Plywood beam
2 Stressed skin panel

using

1 Parallel plies only
2 Full cross section
3 Strength and stiffness

Table A1 : Design values for Canadian Douglas fir $\frac{1}{2}$ in sheathing grade.

Design method	Parallel plies only	Full cross-section	Strength and stiffness
	Stress lbf/in^2	Stress lbf/in^2	Strength per ft width
Bending			
Face grain $\parallel$ span	1500	1188	582 lbf.in
Face grain $\perp$ span	1410	490	240 lbf.in
Tension			
$\parallel$ face grain	1580	948	5631 lbf
$\perp$ face grain	1410	564	3350 lbf
Compression			
$\parallel$ face grain	1110	666	3956 lbf
$\perp$ face grain	860	344	2043 lbf
Rolling shear			
$\parallel$ face grain	50	59.4	245 lbf
$\perp$ face grain	50	32.6	128 lbf
	Modulus of elasticity lbf/in^2	Modulus of elasticity lbf/in^2	Stiffness
Bending			
Face grain $\parallel$ span	1750 000	1386 000	168 000 lbf/in^2
Face grain $\perp$ span	1100 000	228 000	28 000 lbf/in^2
Tension and compression			
$\parallel$ face grain	1750 000	1050 000	6237 000 lbf
$\perp$ face grain	1100 000	440 000	2614 000 lbf

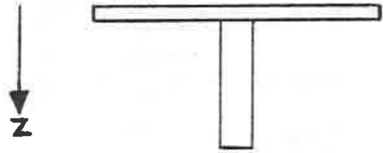
1 Component : Plywood beam (face grain // span) : span $l = 20$ in . width $b = 12$ in . load $w = 50$ lb/ft^2

	Parallel plies only	Full cross-section	Strength and stiffness
2 Design method	Parallel plies only	Full cross-section	Strength and stiffness
3 Geometrical properties	Table 2	Table 13	
Thickness	0.495 in	0.495 in	
Veneer thickness	0.099 in	0.099 in	
Section modulus Z	0.389 in^3	0.490 in^3	
First moment of area Q_{cp}	0.235 in^3	$12 \times 2 \times 0.099 \times 0.1485 = 0.3528$ in^3	
Second moment of area I	0.0961 in^4	0.1213 in^4	
4 Design values	Table A1	Table A1	Table A1
Bending	$\sigma^* = 1500$ lbf/in^2	$\sigma^* = 1188$ lbf/in^2	$M^* = 582$ lbf.in/ft
Rolling shear	$\tau^* = 50$ lbf/in^2	$\tau^* = 59.4$ lbf/in^2	$V^* = 245$ lbf/ft
Elasticity	$E^* = 1750\,000$ lbf/in^2	$E^* = 1386\,000$ lbf/in^2	$E^*I = 168\,000$ lbf/in^2/ft
Deflection	$\Delta^* = 0.003l = 0.060$ in	$\Delta^* = 0.060$ in	$\Delta^* = 0.060$ in
5 Maximum bending moment (M) and shear force (V)			
$M = wl^2/8$	$5 \times 20^2/8 = 250$ lbf.in	250 lbf.in	250 lbf.in
$V = wl/2$	$5 \times 20/2 = 50$ lbf	50 lbf	50 lbf
6 Calculated values			
Bending	$\sigma = \frac{M}{Z} = \frac{250}{0.389} = 643$ lbf/in^2	$\sigma = \frac{M}{Z} = \frac{250}{0.490} = 510$ lbf/in^2	
Rolling shear	$\tau = \frac{VQ_{cp}}{Ib} = \frac{50 \times 0.235}{0.0961 \times 12} = 10.2$ lbf/in^2	$\tau = \frac{VQ_{cp}}{Ib} = \frac{50 \times 0.3528}{0.1213 \times 12} = 12.1$ lbf/in^2	
Deflection	$\Delta = \frac{5wl^4}{384EI} = \frac{5 \times 5 \times 20^4}{384 \times 1.750 \times 10^6 \times 0.0961} = 0.062$ in	$\Delta = \frac{5wl^4}{384EI} = \frac{5 \times 5 \times 20^4}{384 \times 1386\,000 \times 0.1213} = 0.062$ in	$\Delta = \frac{5wl^4}{384EI} = \frac{5 \times 5 \times 20^4}{384 \times 168\,000} = 0.062$ in
7 Ratios of calculated to design values			
Bending	$\frac{\sigma}{\sigma^*} = \frac{643}{1500} = 0.429$	$\frac{\sigma}{\sigma^*} = \frac{510}{1188} = 0.429$	$\frac{M}{M^*} = \frac{250}{582} = 0.429$
Rolling shear	$\frac{\tau}{\tau^*} = \frac{10.2}{50} = 0.204$	$\frac{\tau}{\tau^*} = \frac{12.1}{59.4} = 0.204$	$\frac{V}{V^*} = \frac{50}{245} = 0.204$
Deflection	$\frac{\Delta}{\Delta^*} = \frac{0.062}{0.060} = 1.033$	$\frac{\Delta}{\Delta^*} = \frac{0.062}{0.060} = 1.033$	$\frac{\Delta}{\Delta^*} = \frac{0.062}{0.060} = 1.033$

1 COMPONENT: Stressed skin panel

Span l = 150 in

Load w = 60 lb/ft^2



Plywood skin 12 in x ½ in

(face grain 11 span)

Timber rib 1.5 in x 5.50 in

2 DESIGN METHOD	PARALLEL PLIES ONLY	FULL CROSS-SECTION	STRENGTH AND STIFFNESS
3 GEOMETRICAL PROPERTIES			
Skin: Thickness	0.495 in	0.495 in	0.495 in
Veneer thickness	0.099 in	0.099 in	0.099 in
Area A	3.564 in^2	5.940 in^2	
Second moment I	0.0961 in^4	0.1213 in^4	
Rib: b x d	1.50 x 5.50	1.50 x 5.50	1.50 x 5.50
Area A	8.250 in^2	8.250 in^2	8.250 in^2
Second moment I	20.8 in^4	20.8 in^4	20.8 in^4
4 DESIGN VALUES			
Skin	Table A1	Table A1	Table A1
Panel deflection 0.0031	0.450 in	0.450 in	0.450 in
5 MAXIMUM BENDING MOMENT (M) AND SHEAR FORCE (V)			
$M = wl^2/8$	$5 \times 150^2/8$ = 14062 lbf in	14062 lbf in	14062 lbf in
$V = wl/2$	$5 \times 150/2$ = 375 lbf	375 lbf	375 lbf

2 Design method	Parallel plies only	Full cross-section	Strength and stiffness

6 Location of neutral axis

	Parallel plies only: skin	rib	Σ	Full cross-section: skin	rib	Σ	Strength and stiffness: skin	rib	Σ
E	1.750×10^6	1.500×10^6		1.050×10^6	1.500×10^6			1.500×10^6	
A	3.564	8.250		5.940	8.250			8.250	
EA	6.237×10^6	12.375×10^6	18.612×10^6	6.237×10^6	12.375×10^6	18.612×10^6	6.237×10^6	12.375×10^6	18.612×10^6
z	0.25	3.25		0.25	3.25		0.25	3.25	
EAz	1.559×10^6	40.219×10^6	41.778×10^6	1.559×10^6	40.219×10^6	41.778×10^6	1.559×10^6	40.219×10^6	41.778×10^6

$\bar{z} = \dfrac{\Sigma EA}{\Sigma EAz}$

Parallel plies only: $\bar{z} = \dfrac{41.778\times10^6}{18.612\times10^6} = 2.24$ in

Full cross-section: $\bar{z} = 2.24$ in

Strength and stiffness: $\bar{z} = 2.24$ in

7 Bending stiffness

$EI = \Sigma(EI + EAz^2)$

	EI + EAz² = EI
Parallel plies only	
Skin	$1.750\times10^6\times0.0961 + 1.750\times10^6\times3.564\times1.99^2 = 24.867\times10^6$
Rib	$1.500\times10^6\times20.8 + 1.500\times10^6\times8.250\times1.01^2 = 43.824\times10^6$
	$\underline{68.691\times10^6}$
Full cross-section	
Skin	$1.386\times10^6\times0.1213 + 1.050\times10^6\times5.940\times1.99^2 = 24.867\times10^6$
Rib	$1.500\times10^6\times20.8 + 1.500\times10^6\times8.250\times1.01^2 = 43.824\times10^6$
	$\underline{68.691\times10^6}$
Strength and stiffness	
Skin	$168\,775 + 6.237\times10^6\times1.99^2 = 24.867\times10^6$
Rib	$1.500\times10^6\times20.8 + 1.500\times10^6\times8.250\times1.01^2 = 43.824\times10^6$
	$\underline{68.691\times10^6}$

8 Calculated values

8.1 Plywood

Compression at top of skin

Parallel plies only: $\sigma_c = \dfrac{ME_cz}{EI} = \dfrac{14060\times1.750\times10^6\times2.24}{68.691\times10^6} = 802$ lbf/in²

Full cross-section: $\dfrac{14060\times1.050\times10^6\times2.24}{68.691\times10^6} = 482$ lbf/in²

Strength and stiffness: $\dfrac{14060\times6.237\times10^6\times2.24}{68.691\times10^6} = 2860$ lbf

Rolling shear at critical plane

Parallel plies only: $\tau_{cp} = \dfrac{V\Sigma EQ_{cp}}{b\,EI} = \dfrac{375\times1.750\times10^6\times12\times0.099(2.19+1.99)}{1.50\times68.691\times10^6} = 31.6$ lbf/in²

Full cross-section: $\dfrac{375\times1.050\times10^6\times12\times4\times0.099\times2.04}{1.50\times68.691\times10^6} = 37.0$ lbf/in²

Strength and stiffness: $\dfrac{375\times6.237\times10^6\times\frac{4\times0.099}{5\times0.099}\times2.04}{1.50\times68.691\times10^6} = 37.0$ lbf/in²

8.2 Timber rib

Values are the same for all three methods

Tension stress at bottom of rib $= \dfrac{14060\times1.500\times10^6\times3.76}{68.691\times10^6} = 1155$ lbf/in²

Shear stress at neutral axis $= \dfrac{375\times1.500\times10^6\times1.50\times3.76\times1.88}{1.50\times68.691\times10^6} = 57.9$ lbf/in²

Deflection $= \dfrac{5wl^4}{384EI} = \dfrac{5\times5\times150^4}{384\times68.691\times10^6} = 0.480$ in

9 Ratios of calculated to design values

9.1 Plywood

Compression

Parallel plies only: $\dfrac{\sigma_c}{\sigma_c^*} = \dfrac{802}{1110} = 0.723$

Full cross-section: $\dfrac{\sigma_c}{\sigma_c^*} = \dfrac{482}{666} = 0.723$

Strength and stiffness: $\dfrac{N}{N^*} = \dfrac{2860}{3956} = 0.723$

Rolling shear

Parallel plies only: $\dfrac{\tau}{\tau^*} = \dfrac{31.6}{50} = 0.632$

Full cross-section: $\dfrac{\tau}{\tau^*} = \dfrac{37.0}{59.4} = 0.623$ or $= \dfrac{37.0}{50} = 0.740$

9.2 Timber rib

Stresses and deflections are the same for all three methods

PAPER 2

CIB - WORKING COMMISSION W18

A FRAMEWORK FOR THE PRODUCTION OF AN INTERNATIONAL CODE OF PRACTICE FOR THE STRUCTURAL USE OF TIMBER

by

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COPENHAGEN - OCTOBER 1973

INTRODUCTION

Most developed countries have a national code or standard dealing with the structural use of timber, but the methods of drafting these, and their standing as controlling documents, can be quite different. They often have different objectives and must necessarily relate to the national codes for other materials so that it is unreasonable to expect that their scope and contents will be the same. International harmonisation of codes is certainly desirable and experience with concrete over the past decade has shown that progress can be made, although only slowly.

In the case of concrete, progress towards harmonisation became possible as a result of the work of two international committees, one set up by the International Council for Building (CIB) and the other by the European Committee for Concrete (CEB). The European Committee published its "Recommendations for an International Code of Practice for Reinforced Concrete" in 1963 and later in conjunction with the International Federation for Prestressing, issued "Practical Recommendations for the Design and Construction of Prestressed Concrete Structures" in 1966. These two documents formed the basis for a common approach to the preparation of national standards for concrete and, perhaps unfortunately for timber, set a pattern for the adoption of limit state design.

Although limit state design has not yet been widely adopted, this is likely only to be a matter of time, and it can be expected that codes for timber will also be based on this approach. This raises a number of problems since any change in design procedure must not incur a penalty on section sizes compared with traditional practice, particularly in house construction. Any consideration for a framework for a code for timber should therefore take account of the implications of limit state design.

Other problems which will also have to be considered are the extent to which a code should embrace structural analysis and whether it should deal with test methods, derivation of stresses, performance tests on prototype structures and questions of workmanship, fire resistance and preservation. Obviously the scope of a code, and consideration of its framework, should be kept as wide as possible although some sections could exist and be dealt with separately.

At the present moment the Code of Practice for the structural use of timber in the United Kingdom is being revised. Responsibility for the revision, and indeed for all material codes, is now vested in the British Standards Institution and not as before in the professional institutions. The revision is to be based on limit state design and will include machine stress grading and the new visual grades specified in BS 4978:1973. The procedure has been to set up a main committee with overall responsibility for the framework and final contents of the code with fifteen sub-committees dealing with individual sections. Each sub-committee is chaired by a member of the main committee and may co-opt outside specialists as necessary. This raises problems of co-ordination, particularly in technical matters, and consideration is being given as to how these may be resolved, for example by appointing a full-time liaison officer charged with the drafting of the sub-committees' sections of the Code. The need is to carry through the revision as quickly and effectively as possible while ensuring a sufficiently broad contribution from the timber and construction industries and from the professional institutions.

A framework which has been proposed for the revision of the United Kingdom code, may be divided into the following six major parts, excluding a general introduction giving the scope of the code, the definitions which apply and the symbols used:-

1 Design conditions and material requirements
2 Material properties
3 Joint properties
4 Member design
5 Structural testing
6 Non-structural requirements

DESIGN CONDITIONS AND MATERIAL REQUIREMENTS

This section gives the quality requirements and stress-grades for the materials, and the treatments which may be used, and defines, as far as possible, the loads and limit state conditions which should be taken into account in design. It is divided into the following chapters:-

a DESIGN OBJECTIVES

- Limit State Design
- Limit State Requirements
  - General
  - Ultimate strength
  - Deflection
  - Durability
  - Fire Resistance
  - Vibration
  - Other limit states
- Design Loads
  - General
  - Ultimate strength
  - Deflection

b MATERIALS

- General Requirements
  - Timber
  - Laminated timber
  - Plywood
  - Mechanical fasteners
  - Adhesives
  - Preservatives

c SPECIES OF TIMBER

- Moisture Conditions
  - General
  - Service requirements
  - Strength properties
  - Geometrical properties of sections
  - Temperature

MATERIAL PROPERTIES

This section deals with the individual materials and gives characteristic stresses and modification factors so that design stresses can be derived for the service conditions associated with any particular structure. The section is divided into the following chapters:-

a TIMBER

- General
- Grades
  - Visual
  - Machine
- Characteristic Stresses
- Grade Characteristic Stresses: Visual
  - Timber graded to BS 4978
  - Timber graded to NLGA rules
  - Timber graded to Nordic rules
- Grade Characteristic Stresses: Machine
  - Timber graded to BS 4978
  - Timber graded to other grades
- Modification Factors
  - Size
  - Duration of load
  - Moisture content
  - Load sharing
- Design Stresses
- Other Strength Properties
- Geometrical Properties of Sections

b LAMINATED TIMBER

- General
- Grades
- Tension Laminations
- Modification Factors
  - Grade and number of laminations
  - Curved members
  - Size
  - Duration of load
  - Moisture content
  - Load sharing
- Design Stresses

c PLYWOOD

- General
- Grades
- Grade Characteristic Stresses
  - Standard
  - By test
  - By calculation
- Modification Factors
  - Duration of load
  - Moisture content
- Design Stresses
- Geometrical Properties

Consideration is also being given to the inclusion of blockboard and tempered hardboard in this section; although their application will have to be restricted to limited service conditions.

It is also recognised that because of the state of the plywood industry, and changes in the available species and section arrangements, any tabulated stress data relating to particular specifications could quickly be outdated, and that other specifications could become available. Three approaches have therefore been recommended for plywood; first standard stress tables for the major plywood specifications from Finland and British Columbia, secondly the derivation of stresses from laboratory tests on limited samples and, thirdly, the calculation of stresses from the corresponding strength data for the species used in the plywood. These three approaches should be progressively more conservative in their estimates of stresses.

If this approach is accepted for plywood, and it may be questioned whether the derivation of stresses from test results is appropriate to a code of practice, then it could be applied to other sections.

JOINT PROPERTIES

It is recognised that with the developments which have, and are, taking place in jointing devices it is impossible, within the life span of a Code, to be up to date. This is consequently another section where the inclusion of procedures for deriving loads or stresses from tests could be of advantage. It is also recognised that to include all the jointing devices which are now standardised and adequately specified, would lead to a very lengthy code and as a result the proposal for the United Kingdom Code is that it should include tabulated loads for nails, screws and bolts only, but that a test procedure and analysis should be given which could be applied to other types of joint. This section is divided into the following chapters:-

a DERIVATION OF CHARACTERISTIC LOADS OR STRESSES

- Test Specimens
    - Sampling
    - Preparation
- Test Procedure
- Analysis of Results
    - Strength
    - Deformation
- Characteristic Loads or Stresses
- Deformation

b NAILED JOINTS

- General
- Nail Spacing
- Characteristic Lateral Loads
- Characteristic Withdrawal Loads
- Modification Factors
    - Duration of load
    - Moisture content
    - Nail penetration
    - Number of nails
    - Double shear
    - Metal to wood joints
    - Plywood to wood joints
    - Plywood to plywood joints
- Design Loads
- Design Deformations

c SCREWED JOINTS

- General
- Screw Spacing
- Characteristic Lateral Loads
- Characteristic Withdrawal Loads
- Modification Factors
  - Duration of load
  - Moisture content
  - Screw penetration
  - Number of screws
  - Metal to wood joints
  - Plywood to wood joints
  - Plywood to plywood joints
- Design Loads
- Design Deformations

d BOLTED JOINTS

- General
- Bolt Spacing
- Characteristic Lateral Loads
- Modification Factors
  - Duration of load
  - Moisture content
  - Angle of grain
  - Multiple shear
  - Number of bolts
  - Metal to wood joints
  - Plywood to wood joints
  - Plywood to plywood joints
- Design Loads
- Design Deformations

e GLUED JOINTS

- General
  - Preparation
  - Fabrication
- End Joints
  - Scarf
  - Finger
- Side Joints
  - Splice plates
- Characteristic Stresses
- Modification Factors
  - End joints
  - Side joints
- Design Stresses

MEMBER DESIGN

This section deals with the design of individual structural members and frames, where special modification factors may apply, and its extent will obviously depend on how far a Code of Practice may be required to deal with the subject of design analysis generally. The section is divided into the following sections:-

a Flexural Members
b Compression Members
c Tension Members
d Members Subject to Combined Stresses
e Structural Frames
f Floor and Roof Boarding

STRUCTURAL TESTING

In the United Kingdom it is accepted that a structure may be shown to be adequate on the basis of design calculations or from the results of tests on full-size prototypes. There are various reasons why one course rather than the other may be preferred, or indeed even be necessary, but the Code should provide the opportunity to adopt either, subject to agreement between the parties concerned. Although there is a need for research on the subject of prototype testing, on load factors, time effects, variability, and acceptance criteria there is now a considerable experience with this approach to permit its inclusion in a Code. This section is divided into the following chapters:-

a PROTOTYPE TESTING
- General
- Test Samples
- Test Procedure
  - Deflection
  - Ultimate strength
- Test Loads
- Performance Requirements
  - Deflection
  - Ultimate strength

b QUALITY CONTROL
- General
- Test Samples
- Test Procedure
- Test Loads
- Performance Requirements

The factory prefabrication of relatively large batches of standard structural components, such as trussed rafters and floor and roof panels, is a growing feature of timber construction. Quality control over fabrication and selection of materials is of considerable importance, and this may be approached by inspection or testing, or by a combination of both. Quality control, by testing production units might therefore become a necessary part of a code, and it is for this reason that chapter (b) has been included.

NON-STRUCTURAL REQUIREMENTS

The final section of the Code deals with those aspects of construction which, although not part of engineering design as such, nevertheless can have a significant influence on performance and on ensuring the acceptance of a design in relation to other safety requirements. In addition this section gives general advice on matters which are peculiar to timber and timber structures and which have a bearing on performance. The section contains the following chapters:-

a Workmanship
b Preservation
c Fire Resistance
d Inspection
e Maintenance

CONCLUSIONS

It should be pointed out that the outline of the contents for a Code given above is one that has been suggested for use in the United Kingdom but has not as yet been formally adopted. Although the general arrangement and order of chapters follows a consistent plan there are probably many equally acceptable alternatives, and certainly other arrangements have been employed in other codes. It is believed however that the chapters adequately cover the subject and perhaps the next task should be to survey existing codes so that the chapters may be expanded along common lines.

PAPER 3

CIB - WORKING COMMISSION W18

THE DESIGN OF SOLID TIMBER COLUMNS

by

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1. SYNOPSIS

This report has been prepared as the basis for discussions in CIB Working Group W 18 regarding the possibilities of formulating on a uniform technical basis Codes for Timber Structures, so that differences only arise due to different loading and safety levels.

The report has been prepared on the basis of standards and supplementary material received from participants in the Working Group. The most important material is listed in section 7.

So far, only solid columns have been dealt with, inter alia, because it is relatively easy - at any rate theoretically - to expand the calculation principles for solid columns to apply to composite columns as well.

As repards specification of assumptions etc., these have been restricted to essentials, readers being requested to show a reasonable degree of goodwill - for example, it is not explicitly stated that the expression for bending stresses (Moment/Moment of resistance) assumes the use of principal axis.

2. THEORY FOR CENTRALLY LOADED COLUMNS

2.1 Perfect columns

For centrally loaded, straight, homogeneous columns with constant cross-section and of linear-elastic material, the bearing capacity is limited by the expressions

$$\sigma_c/s_c \leq 1 \tag{1}$$

and

$$\sigma_c/s_c \leq k_E = s_E/s_c = \frac{\pi^2 E}{\lambda^2 s_c} \tag{2}$$

where

σ_c is the compression stress. $\sigma_c = P/A$, where

P is the normal force and

A is the sectional area.

s_c is the compressive strength.

s_E is the Euler stress.

E is the modulus of elasticity in compression.

λ is the slenderness ratio. $\lambda = l/i$, where

l is the free length and

i is the radius of gyration.

(1) and (2) specify an upper bound for the bearing capacity, in that the eccentricities, inhomogeneities, deviations from straightness, etc. occurring in practice result in a reduction in the bearing capacity.

2.2 Eccentrically loaded columns with an initial deflection

A more realistic calculation is obtained by considering an eccentrically loaded column with an initial deflection.

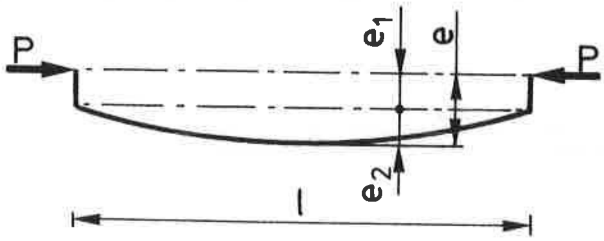

Fig. 1

At the middle, the normal force P acts with the eccentricity $e = e_1 + e_2$, see fig. 1.

e_1 takes account of the fact that the load is unavoidably applied with a certain eccentricity, inter alia because inhomogeneities mean that the geometrical and elastic centres do not coincide.

e_2 takes possible initial deflection into account.

Normally, e_1 is taken to be independent of the length, while e_2 is taken as proportional to the length.

It has proved practical to specify the eccentricity relative to the core radius k corresponding to the direction of deflection.

This relative eccentricity is denoted ϵ, and it is thus normally assumed - for the reasons given above - that

$$\epsilon = e/k = \epsilon_1 + \epsilon_2 = a + b\lambda \tag{3}$$

where a and b are constants.

For non-symmetrical cross-sections, the core radius corresponding to the compression side is used, i.e. $k = W/A$, where W is the corresponding section modulus.

Because of the normal force, the column will deflect, and the eccentricity e at the middle be increased. The final eccentricity is denoted u.

Still assuming that the material is linear-elastic and that e_2 varies sinusoidally, and applying the usual technical theory of elasticity, we find that

$$\text{for } e_1 = 0: \quad u = e \frac{k_E}{k_E - \frac{\sigma_c}{s_c}} \tag{4}$$

$$\text{for } e_2 = 0: \quad u = \frac{e}{\cos\left(\frac{\pi}{2}\sqrt{\frac{\sigma_c}{s_E}}\right)} \sim e \frac{k_E + 0{,}25\frac{\sigma_c}{s_c}}{k_E - \frac{\sigma_c}{s_c}} \tag{5}$$

As an approximation, (4) is normally used in all cases, and this will also be done in the following.

From the normal force P, we get the normal stress $\sigma_c = P/A$.

The moment Pu acting at the middle gives on the compression side a maximum normal stress of σ_b

$$\sigma_b = Pu/W = Pu/(kA) = \sigma_c \frac{u}{k} \tag{6}$$

According to normal practice, the following condition must be satisfied, the equality sign corresponding to exhaustion of the bearing capacity:

$$\frac{\sigma_c}{s_c} + \frac{\sigma_b}{s_b} \leq 1 \qquad (7)$$

where s_b is the bending strength.

However, this criterion is not immediately applicable. In the determination of s_c the load is assumed to act centrally, while here, cf. (3), a bending stress of the magnitude $Pe_1/W = Pak/(kA) = a\sigma_c$ is assumed for $\lambda = 0$.

It is therefore necessary to correct (7) to

$$(1 - a\,\frac{s_c}{s_b})\frac{\sigma_c}{s_c} + \frac{\sigma_b}{s_b} \leq 1 \qquad (8)$$

With

$$\beta = s_c/s_b \qquad (9)$$

and

$$\psi = 1 - a\,\frac{s_c}{s_b} \qquad (10)$$

we find from (8) by insertion of (4):

$$\psi\,\frac{\sigma_c}{s_c} + \beta\,\frac{\sigma_c}{s_c}\,\epsilon\,\frac{k_E}{k_E - \frac{\sigma_c}{s_c}} \leq 1 \qquad (11)$$

from which we get

$$\frac{s_{cr}}{s_c} = \frac{1 + (\psi + \beta\epsilon)k_E}{2\psi} - \sqrt{[\frac{1 + (\psi + \beta\epsilon)k_E}{2\psi}]^2 - \frac{k_E}{\psi}} \qquad (12)$$

the values of σ_c corresponding to the equality sign in (11) being denoted s_{cr}(cr = critical).

3. COLUMN DESIGN CURVES OF DIFFERENT COUNTRIES

3.0 General

In the following the codified column calculations of various countries are reported and compared.

As the bearing capacity depends on both E and s_c, it is necessary - in order to obtain comparable values - to select a ratio for E/s_c. In the following, $E/s_c \sim 300$ is assumed, which presumably corresponds to the ratio between the characteristic short-term values for the commonest types of timber.

In a number of cases, no basis for calculation is given, only the permissible stresses being specified. In these cases, minor adjustments may have been made in order to obtain comparable values.

This section and section 4 are based on characteristic values; reparding the specification of safety factors, reference is made to section 5. Various safety factors are often - quite inconsistently - included in the standard column expressions. For example, in the UK, permissible values are used for s_c and "minimum values" for E.

3.1 United Kingdom, South Africa, Western Germany and Holland.

3.1.0 General.

The section on columns in the standards of these countries is based on formula (12), although different values are used for the parameters involved, especially ϵ.

3.1.1 United Kingdom (CP 112:1967 [12.1])

In (12), which in UK is denoted the Perry-Robertson formula, $\beta = 1$ is used (even though it is otherwise assumed that $s_c/s_b < 1$) and $\epsilon = b\lambda$, where b lies between 0.001 and 0.003. It is thus assumed that a = 0, which leads to $\psi = 1$ (formula (10)).

b = 0.002 corresponds to a camber of about 1/900 at the middle of a column of length l, a value that appears unrealistic compared with what is considered acceptable pursuant to the grading rules for structural timber.

The choice of b is based partly on tests carried out by Robertson [12.2] and Sunley [12.4], but it looks as though the writers of the standard have overlooked the fact that the tests were carried out on specimens that were "... surfaced on all four faces to remove any bow or twist ..." [12.4]. The ϵ-values applied thus correspond only to inhomogeneities etc., but not to the initial deflections which are permitted in structural timber.

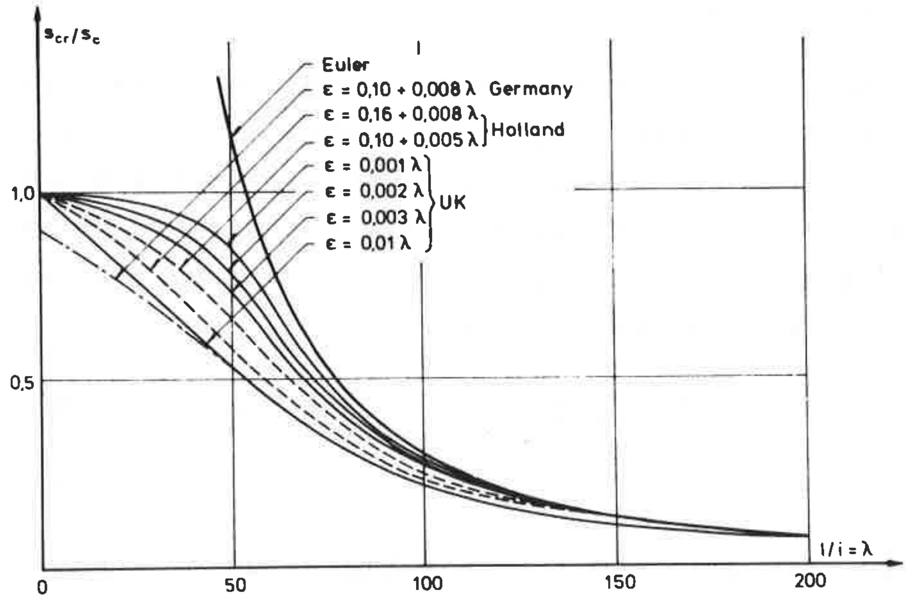

Fig 2

In fig. 2, s_{cr}/s_c is shown in relation to λ for b = 0.001, 0.002 and 0.003 and - as a more realistic value - 0.01.

3.1.2 South Africa

The present draft [10.1] for a revision corresponds, in principle, to CP 112:1967.

3.1.3 West Germany (DIN 1052, 1969 [6.1])

It is assumed that $\beta = 1$, although in the other sections, $\beta < 1$, and that $\epsilon = 0.10 + 0.008\lambda$. Even though it is assumed that $a \neq 0$, the standard still sets $\psi = 1$, which results in inconsistency at the limit $\psi = 0$. However, this is concealed later in the fixing of the factors of safety.

s_{cr}/s_c are also shown in fig. 2.

3.1.4 Holland (NEN 3852, July 1973 [7.1])

It is assumed that $\beta = 0.75$ and that

$$\epsilon = 0.16 + 0.008\lambda \tag{13}$$

for the lower grade (standardbouwhout) and that

$$\epsilon = 0.10 + 0.005\lambda \tag{14}$$

for the higher grade (constructiehout).

As far as can be seen, these values have been fixed on the basis of an earlier German proposal. In the final German rules, cf. 3.1.3, a combination of (13) and (14) is used for both grades.

Calculating correctly with the ψ-value appearing from (10), s_{cr}/s_c is found as shown in fig. 2.

The Dutch standard includes a further requirement, which is formulated in a very fine and complicated style as a limitation on deflection, but this really only states that s_{cr}/s_c must not be put higher than $0.6k_E$ for standardbouwhout and $0.75k_E$ for constructiehout. There seems to be no rational motivation for this rule, which is not included in fig. 2.

3.2 Brazil, Canada, USA, Denmark

In these countries, the calculation is, in principle, based on the formula of Engesser or Southwell [12.3]:

$$s_{cr} = \frac{\pi^2 E'}{\lambda^2} \tag{15}$$

where E' is the modulus of elasticity, which is assumed to depend on the normal stress σ_c. For the normal stresses in the range zero to the limit of elasticity, it is assumed that $E' = E$, i.e. (15) and (2) are identical. If it is assumed that the stress-strain curve has horizontal tangent for $\sigma_c = s_c$, then (1) is also automatically contained in (15).

In the theory, (15) is thus based on the assumption that the deviation from the perfect Euler curve is due solely to the fact that the material is not linear elastic, while the eccentricities, inhomogeneities, etc. are not taken into account explicitly. In practice, however, certain account is taken of these, in that the stress-strain curves used are considerably less favourable than those obtained in tests with the materials. In fact, the impression received is that the column curve is chosen first, possibly on the basis of tests, after which the stress-strain curve giving the desired result is determined.

Column curves of this nature are applied in many countries, including Brazil [2.1], Canada [3.1] and parts of the USA (on the basis of tests by Madison [13.2]), Denmark [4.1] (inter alia on the basis of [4.2])and the East-European countries. Similar curves were previously also used in France.

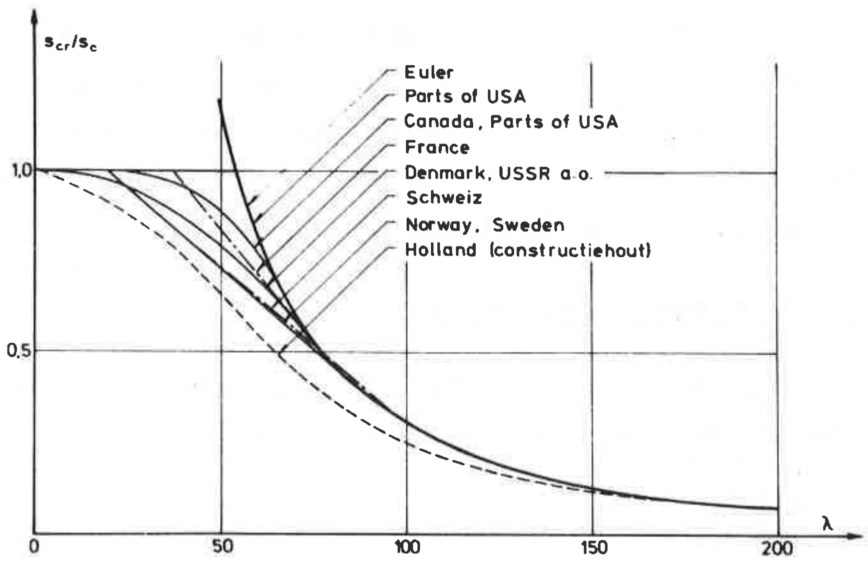

Fig. 3

As the theoretical basis is primitive, and as objections can be made to the basic tests in many cases, the theoretical foundation is not given; instead, the column curves are simply drawn in on fig. 3, together with the Dutch curve corresponding to constructiehout for the purposes of comparison.

The objections to the tests relate partly to the test procedure, in which efforts are often made to eliminate the effect of inhomogeneities, and partly to the test material, which has in many cases been of a considerably higher quality that that used in practice.

The column curve shown in [13.1], which is used in parts of the USA, can also be classified as this type: It is assumed that s_c and the limit of elasticity coincide, i.e. the standard reckons on the Euler curve right up to $s_{cr} = s_c$. That this is an unjustifiable assumption was recognized by every other country at about the turn of the century. As "compensation" for the defective theoretical basis, a precisely specified safety factor is used, namely, 2.727.

3.3 France, Norway, Sweden, Switzerland and USA

On the basis of, inter alia, Tetmajer's tests [9.2], [9.3], a number of other countries, including France [5.1], Norway [8.1], Sweden [11.1] and Switzerland [9.1], have used a column curve consisting of the Euler curve and, for shorter columns, a straight line, frequently tangential to the Euler curve.

In accordance with Tetmajer's original proposal, a straight line through $(\lambda, s_{cr}/s_c) = (0 , 1)$ was adopted, but later, straight lines were chosen, corresponding to $s_{cr}/s_c = 1$ for λ-values below 20-40, cf. fig. 3, where the column curves of the above countries are also shown.

4. COMPRESSION AND BENDING

4.1 Theory

The following is based on the assumption that the moment is zero at the ends and varies sinusoidally, but the expressions derived are normally applies generally, cf. the remarks in connexion with (4) and (5).

If the normal stresses at the middle from the moment are σ_b, this corresponds to the normal force acting with the relative eccentricity σ_b/σ_c in addition to the eccentricities considered earlier, which are due to initial deflection, inhomogeneities, etc. By substituting $\epsilon + \sigma_b/\sigma_c$ for ϵ, (11) can be used directly to determine the precisely acceptable, related values of σ_c/s_c and σ_b/s_b.

We find that

$$\frac{\sigma_c}{s_c} = \frac{1 + (\psi + \beta\epsilon)k_E}{2\psi} - \sqrt{\left(\frac{1 + (\psi + \beta\epsilon)k_E}{2\psi}\right)^2 - \left(1 - \frac{\sigma_b}{s_b}\right)\frac{k_E}{\psi}} \quad (16)$$

Here, it is assumed that the compression zone is decisive for the bearing capacity.

In the case of very asymmetrical cross-sections, the tensile zone may be decisive, and one would think that this could be allowed for by altering (8) to

$$-\frac{\sigma_c}{s_c} + \frac{\sigma_b}{s_b} \leq 1 \quad (17)$$

with σ_c still assumed to be positive as compression. However, according to [4.3], this may lead to an unreasonably high resultant stress σ_r.

As

$$\sigma_r = \sigma_b - \sigma_c \quad (18)$$

we get, when (17) is satisfied,

$$\frac{\sigma_r}{s_b} = 1 + \frac{\sigma_c}{s_c} - \frac{\sigma_c}{s_b} = 1 + \sigma_c \frac{s_b - s_c}{s_c \; s_b} \quad (19)$$

As $s_b > s_c$, it will be seen that the resulting stress is greater than s_b, which appears unreasonable.

In the following, therefore, instead of (17), we will use the criterion

$$\sigma_b - \sigma_c \leq s_b \quad (20)$$

By calculations analogous to those leading to (16), we find that the precisely acceptable, related values of $\frac{\sigma_c}{s_c}$ and $\frac{\sigma_b}{s_b}$ are given by

$$\frac{\sigma_b}{s_b} = (1 + \beta\frac{\sigma_c}{s_c}) \frac{k_E - \frac{\sigma_c}{s_c}}{k_E} - \epsilon\beta \frac{\sigma_c}{s_c} \qquad (21)$$

The bearing capacity is, of cource, also limited by $\sigma_c \leq s_{cr}$.

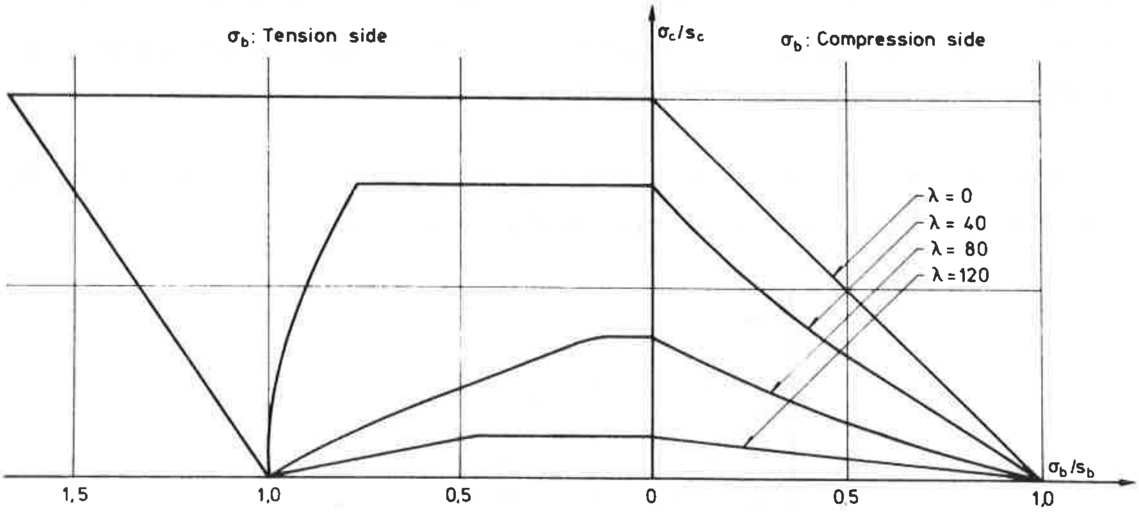

Fig. 4

If, for example, we take the values corresponding to the Dutch constructiehout, i.e. $\beta = 0.75$ and $\epsilon = 0.10 + 0.005\lambda$, and once more assume that $E/s_c \sim 300$, we arrive at the acceptable stress combinations shown in fig. 4. There is one slight inconsistency in the calculations in that the same value of ϵ is used when calculating according to (16) as to (21), even though ϵ in (16) is assumed to be relative to the core radius of the compression side, whereas, in (21) it is assumed to be relative to the core radius of the tension side.

4.2 Approximation

In the standards of practically all countries, it is simply required that

$$\frac{\sigma_c}{s_{cr}} + \frac{\sigma_b}{s_b} \leq 1 \qquad (22)$$

It will be seen from fig. 4 that this is a good approximation provided the compression side is decisive - which is almost always the case. The expression is senseless in the exceptional cases in which the tensile side is decisive.

In (22), s_{cr} is normally assumed to correspond to the direction of loading, although in West Germany, the standards requires that the minimum value for s_{cr} be applied. The reason given for this is that it would otherwise be necessary - on account of the low torsional strength of wood - to investigate combined bending and torsional deflection.

In Brazil, a slightly different interpolation formula is applied, but the results deviate only slightly from (22).

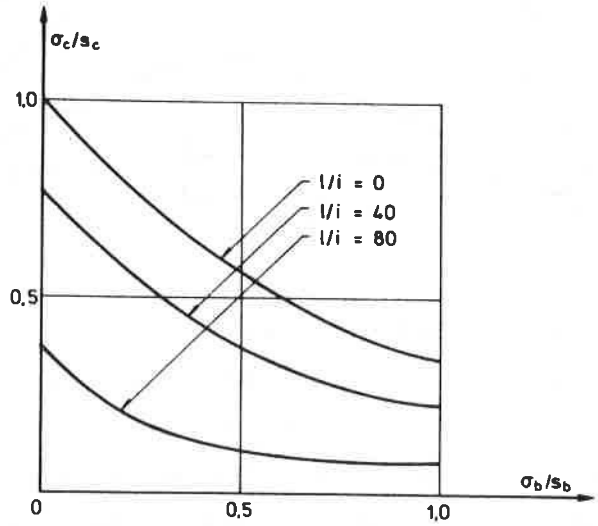

Fig. 5

In Switzerland, (22) is substituted by the following criterion:

$$\frac{\sigma_c}{s_c} \le \frac{\beta \dfrac{\sigma_c}{s_c} + 0.15 \dfrac{\sigma_b}{s_b}}{\beta \dfrac{\sigma_c}{s_c} + \dfrac{\sigma_b}{s_b}} \cdot \frac{s_{cr}}{s_c} \qquad (23)$$

This is shown in fig. 5, with values corresponding to fig. 4. The result seems unreasonable; in particular, it should be noted that the ordinary bending criterion is not included as a boundary case.

5. INTRODUCTION OF THE SAFETY FACTOR

If characteristic values, corresponding for example to the

1% or 5% fractile, are used for E and s_c in the expressions derived, the bearing capacity for all λ-values will be determined with the same accuracy, and there will therefore be no reason not to use the same safety factor in all cases.

In most cases, the present standards contain rather arbitrary variations of the factors of safety.

In certain cases, increased safety factors are applied for the slender columns, even though the determination of the bearing capacity of such columns is very reliable; regardless of the magnitude of eccentricities, etc., the bearing capacity corresponds to the Euler bearing capacity. As an example of an increased safety factor of this type, the Dutch requirements can be mentioned, which lay down a factor of safety of 3.6 against exceeding 60-75% of the Euler stress.

In other cases, increased safety factors are used for short columns because s_c varies more than E. Such an increased factor, used, inter alia, in West Germany, is justified when based on the mean values of the material values, but not when uniformly fixed characteristic values are applied.

6. LIMITATION OF λ

1. Brazil : 140
2. Canada : None
3. Denmark : 200
4. France : 200?
5. Germany : 150 (200 for secondary constructions)
6. Holland : 200?
7. Norway : 170?
8. Switzerland: 150 (120 for bridge construction)
9. UK : 180, although 250 if wind is the only load
10. USA : 170

"?" indicates that the limitation is not formulated explicitly but appears from the extent of tables or diagrams.

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